PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXX. No. 6.

AUGUST, 1904.

Edited by the Secretary, under the direction of the Committee on Publications.

CONTENTS.

NEW YORK 1904.

American Society of Civil Engineers.

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Term expires January, 1905:	*Term expires January, 1906:*
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THE PRESIDENT OF THE SOCIETY IS *ex-officio* MEMBER OF ALL COMMITTEES.

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The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - - - 583 Columbus.
CABLE ADDRESS, - - - - - "Ceas, New York."

Vol. XXX. AUGUST, 1904. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

June 1st, 1904.—The meeting was called to order at 8.40 P. M.; James Owen, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 103 members and 11 guests.

The minutes of the meetings of May 4th and 18th, 1904, were approved as printed in the *Proceedings* for May, 1904.

A paper, "On Sedimentation" by Allen Hazen, M. Am. Soc. C. E., was presented by the author.

A written discussion from G. W. Pearsons, M. Am. Soc. C. E., was presented by the Secretary, and the paper was discussed verbally by Messrs. George W. Fuller, George A. Soper, R. S. Weston and the author.

Ballots for membership were canvassed, and the following candidates elected:

As Members.

Edmund Hamilton Bowser, Slidell, La.
St. John Clarke, Bogota, N. J.
Frank Winslow Conn, Philadelphia, Pa.
Clark Dillenbeck, Philadelphia, Pa.
Frederic Clark Dunlap, Philadelphia, Pa.
Emory Alexander Ellsworth, Holyoke, Mass.
Alonzo John Hammond, South Bend, Ind.
John Elden Palmer, Boston, Mass.
Frank Julian Sprague, New York City.
Otto Bruno Suhr, Niagara Falls, Ont., Canada.

As Associate Members.

Charles Metcalf Allen, Worcester, Mass.
Charles Webster L Armour, Fort Smith, Ark.
John Henry Best, Peoria, Ill.
Percival Mitchell Churchill, Denver, Colo.
Park Andrew Dallis, Greenville, S. C.
John Lea Dillard, Boxley, W. Va.
Charles Henry Farnham, Canton, China.
James Easton Ferguson, Detroit, Mich.
Mortimer Foster, New York City.
Charles Rice Gow, Boston, Mass.
George Scott Hubbell, Hoboken, N. J.
Thomas John Jones, Keswick, Cal.
Henry James MacNair, Cleveland, Ohio.
Ralph Barton Manter, Canton, China.
Ernest George Matheson, New York City.
Frederic Antes Snyder, Summit, N. J.

As Associates.

Basil Henry Leather, New York City.
John Joseph Monahan, Boston, Mass.

The Secretary made the following announcements:

The transfer of the following candidates, by the Board of Direction, on May 31st, 1904:

FROM ASSOCIATE MEMBER TO MEMBER.

MORTIMER GRANT BARNES, Geneseo, Ill.
GEORGE WARREN FULLER, New York City.
DUNKIN WIRGMAN HEMMING, New York City.
FREDERICK WILLIAM HONENS, Sterling, Ill.
ERNEST BURSLEM THOMSON, Miami, Fla.
JAMES LAWRENCE TIGHE, Holyoke, Mass.

The election of the following candidates, by the Board of Direction:

AS JUNIORS.

On May 3d, 1904,
EDWARD CHRISTIAN DICKE, St. Louis, Mo.
On May 31st, 1904,
JOSEPH MANUEL BABÉ, Santa Clara, Cuba.
WILLIAM JAMES BACKES, West Haven, Conn.
JOHN ROSS BATES, Brooklyn, N. Y.
ERNEST WILLARD CRAWLEY, New Haven, Conn.
JOHN MIFFLIN HOOD, Jr., Baltimore, Md.
EDWIN LORING SPRAGUE, Jr., Brooklyn, N. Y.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

May 31st, 1904.—8.50 P. M.—Vice-President Deyo in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Craven, Croes, Jackson, Knap, Noble, and Osgood.

The following Committee was appointed to arrange for the Annual Convention, which is to be held at the time of the International Engineering Congress at St. Louis, October 3d to 8th, 1904: George H. Pegram, Alfred Craven, Joseph O. Osgood, George S. Davison, Hunter McDonald and Chas. Warren Hunt.

C. C. Schneider was appointed Chairman of a Committee on Concrete and Steel-Concrete. The Committee as appointed is made up as follows: C. C. Schneider, J. E. Greiner, W. K. Hatt, Olaf Hoff, Robert W. Lesley, J. W. Schaub, Emil Swensson, A. N. Talbot, J. R. Worcester.

On motion, the following resolution was adopted:

"Resolved, That letter-ballots for admission to the Society shall hereafter contain the following information only in regard to each candidate:

1. His name.
2. Date and place of his birth.
3. Date and place of graduation (if any), with degree conferred.
4. Present title (if any) and address.
5. A reference to the Blue List on which his record is given in full."

The following resolution was passed:

"Resolved, That for remittances mailed after July 1st, 1904, the cost of membership in the International Engineering Congress be increased to $10, and said subscription to membership must be received before October 1st, 1904."

The resignation of Robert J. Pratt, Assoc. Am. Soc. C. E., was accepted.

Applications were considered and other routine business transacted. Six Associate Members were transferred to the grade of Member, and six candidates for Junior were elected.

Adjourned.

ANNOUNCEMENTS.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

MEETINGS.

Wednesday, September 7th, 1904.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and two papers will be presented for discussion, as follows: "The Installation of a Pneumatic Pumping Plant," by Arthur H. Diamant, Jun. Am. Soc. C. E., and "Some Notes on the Creeping of Rails," by Samuel Tobias Wagner, M. Am. Soc. C. E.

Both these papers were printed in *Proceedings* for May, 1904.

Wednesday, September 21st, 1904.—8.30 P. M.—At this meeting three papers will be presented for discussion, as follows: "General Methods for the Calculation of Statically Indeterminate Bridges, as used in the Check Calculations of Designs for the Manhattan Bridge and the Blackwell's Island Bridge, New York," by Frank H. Cilley, S. B.; "A Rational Form of Stiffened Suspension Bridge," by Gustav Lindenthal, M. Am. Soc. C. E.; and "Theory and Formulas for the Analytical Computation of a Three-Span Suspension Bridge with Braced Cable," by Leon S. Moisseiff, Assoc. M. Am. Soc. C. E.

These three papers are printed in this number of *Proceedings.*

INTERNATIONAL ENGINEERING CONGRESS AND THIRTY-SIXTH ANNUAL CONVENTION OF THE SOCIETY.

As previously announced, in the Programme of the **International Engineering Congress**, which has been issued to all members of the Society, there will be meetings of the **Congress** at 10 A. M. each day, beginning Monday, October 3d, and continuing throughout the week, the last meeting of the Congress being held on Saturday morning, October 8th. **Members of the Society in all grades are Members of the Congress.**

The **Thirty-Sixth Annual Convention** of the **Society** will be held on the afternoon of Monday, October 3d, in the Hall of Congresses, Louisiana Purchase Exposition, at 2.30 P. M. At this meeting the President will deliver the Annual Address. At the close of the address of the President, the business meeting required by the Constitution will be held. A meeting of the Board of Direction, as required by the Constitution, will be held at a time to be subsequently determined.

A circular, relating to hotel accommodations, which was prepared by the Local Committee of the Society in St. Louis, has been issued to all members, and in this the "Inside Inn" was recommended as, all things considered, the most available for members and their families.

In this circular it is explained that it is impossible to secure a lower rate for transportation than the Exposition excursion rate established by all railroads, this rate being less than the usual, one fare and one-third, round trip, convention rate.

To all members of the Society who have, in response to the circular already issued, specified the subjects it is their purpose to discuss, copies of such of the papers as have been printed in advance form have already been forwarded, and, upon request, the Secretary will be glad to forward such copies to members who have not already asked for them.

Owing to the large number of papers, which cover many subjects, it will be impossible to forward Advance Copies of all papers to all members of the Congress, and it has been necessary to restrict the issue of Advance Copies to those who expect to present discussions either in person at the Congress or by letter.

INTERNATIONAL ENGINEERING CONGRESS.

CHAIRMAN:

Charles Hermany, President, Am. Soc. C. E.

CHAIRMEN OF SECTIONS:

Section A.—Waterways.
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Section B.—Municipal.
J. James R. Croes, Past-President, Am. Soc. C. E.

Section C.—Railroads.
Robert Moore, Past-President, Am. Soc. C. E.

Section D.—Materials of Construction.
Frederic P. Stearns, M. Am. Soc. C. E.

Section E.—Mechanical.
William Metcalf, Past-President, Am. Soc. C. E.

Section F.—Electrical.
Frank J. Sprague, M. Am. Soc. C. E.

Section G.—Military and Naval.

WILLIAM P. CRAIGHILL, Brig.-Gen. U. S. A. (retired);
Past-President, Am. Soc. C. E.

Section H.—Miscellaneous.

OCTAVE CHANUTE, Past-President, Am. Soc. C. E.

SECRETARY:

CHARLES WARREN HUNT, *Secretary*, Am. Soc. C. E.

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GEORGE S. DAVISON, Pittsburg, Pa.

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JOSEPH O. OSGOOD, HUNTER McDONALD,
CHAS. WARREN HUNT, *Secretary*.

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C. W. Buchholz,
Theodore Cooper,
Foster Crowell,
S. L. F. Deyo,
G. J. Fiebeger,
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C. E. Grunsky,
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Rudolph Hering,
Clemens Herschel,
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J. Waldo Smith,
Wm. H. Wiley,
W. J. Wilgus,
Chas. Warren Hunt, *Secretary.*

UNIVERSAL EXPOSITION, ST. LOUIS, 1904.

The Society has undertaken to provide for an engineering exhibit, and the establishment of Headquarters for visiting engineers **in the center of the Liberal Arts Building,** and the Board of Direction has appropriated sufficient funds to defray the necessary expense.

This matter is in the hands of the following Committee:

Robert Moore, M. Am. Soc. C. E., St. Louis, Mo., *Chairman.*
Edward C. Carter, M. Am. Soc. C. E., Chicago, Ill.
Mordecai T. Endicott, M. Am. Soc. C. E., Washington, D. C.
James L. Frazier, " " Frankfort, Ind.
William Jackson, " " Boston, Mass.
Emil Kuichling, " " New York, N. Y.
J. L. Van Ornum, " " St. Louis, Mo.
John F. Wallace, " " Chicago, Ill.
O. E. Mogensen, *Sec'ty,* " " St. Louis, Mo.

PRIVILEGES OF LOCAL SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

The **Boston Society of Civil Engineers** will welcome any member of the American Society of Civil Engineers at its library and reading room, 715 Tremont Temple, Boston, which is open on week days from 9 A. M. to 5 P. M. Members will also be welcome at the meetings, which are held in the same building on the evenings of the fourth Wednesday in January, and the third Wednesdays of other months, except July and August.

The rooms of the **St. Louis Engineers' Club,** in the business center of St. Louis, will be kept open during the World's Fair season, May 1st to December 1st, 1904, and visiting engineers are cordially invited to use them for mail, telephone service, information, etc.

The courtesies of the **Engineers' Society of Western Pennsylvania** have been extended to members of the American Society of Civil Engineers. The rooms of the Society, 410 Penn Ave., Pittsburg, Pa., are open at all times, and meetings are held as follows, except during July and August. REGULAR SECTION, Third Tuesdays; CHEMICAL SECTION, Thursdays following third Tuesdays; MECHANICAL SECTION, first Tuesdays; STRUCTURAL SECTION, Fourth Tuesdays.

The Western Society of Engineers, Monadnock Block, Chicago, Ill., tenders to members of this Society the use of its rooms and facilities, together with the good offices of its Secretary and of a special committee appointed for that purpose.

The Civil Engineers' Club of Cleveland, Ohio, invites members of this Society to make use of the Club rooms, at any time when in Cleveland. Cards will be furnished on application to the Secretary, Mr. J. C. Beardsley.

SEARCHES IN THE LIBRARY.

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many such searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling, compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

Copies of all lists of references are filed, so that in many cases it is only necessary to make a typewritten copy, which reduces the cost of searches to a minimum.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

ACCESSIONS TO THE LIBRARY.

From May 11th to August 6th, 1904.

DONATIONS.*

GENERAL INDEX TO THE STREET RAILWAY JOURNAL BY SUBJECTS AND AUTHORS.

Oct., 1884, to Dec., 1903, Including Vols. I–XXII. Cloth, 10 x 8 ins., 162 pp. New York, McGraw Publishing Company, 1904. $5.00.

This index covers the articles in the American edition of the *Street Railway Journal*. The differences in the International edition are so great that the index would be practically useless if applied to that. No attempt has been made to preserve the original captions of the articles. Except in the author's index, the index is one of subjects, not of titles, and in a long article, where different topics have been treated at length, separate entries of these different topics have been made as far as possible. In addition, all articles susceptible of geographical designation have been indexed under the name of the city or country to which the subject has pertained. For this reason, it is thought that the entries under each city will comprise practically all the references in the paper to events and apparatus in that city. In case of nearly all the cities the articles are indexed independently of the names of the street railway companies, except where they refer to corporate events, such as a change in officers, annual reports, etc. As far as possible, an article or topic has been entered under one heading only, but it is thought that by the system of cross references adopted any important article can be found readily by a little search.

ELECTRICITY AND MATTER.

By J. J. Thomson. Cloth, 8 x 5 ins., 162 pp., illus. New York, Charles Scribner's Sons, 1904.

In these lectures, given at Yale University in May, 1903, the author has attempted to discuss in a simple and untechnical manner the bearings of the recent advances made in electrical researches upon the views of the constitution of matter and the nature of electricity; two questions which in the author's opinion, are so intimately connected, that the solution of one would supply that of the other. A characteristic feature of recent electrical researches, such as the study and discovery of Cathode and Röntgen rays and radio active substances, has been the degree in which they have involved the relation between matter and electricity. The contents are: Representation of the Electric Field by Lines of Force; Electrical and Bound Mass; Effects Due to the Acceleration of Faraday Tubes; The Automatic Structure of Electricity; The Construction of the Atom; Radio-Activity and Radio-Active Substances.

LES ACIERS SPECIAUX.

Aciers au Nickel, Aciers au Manganèse, Aciers au Silicium. Par Léon Guillet. Préface de Henry Le Chatelier. Paper, 11 x 9 ins., 4 + 100 pp., illus. Paris, Vve. Ch. Dunod, 1904. 10 francs.

The author has attempted, in this work, to show the recent industrial advancement and the progress in the manufacture of steel. He has himself made a number of experiments with special samples of steel and in this work he gives the results and also shows the composition and treatment of different kinds of steel. This book is intended as a study of the micrography of nickel, manganese and silicon steel. It contains the analyses and diagrams of tests of various kinds for steel. An account is given of the conditions under which these various alloys may be used.

FORMULAIRE DES CENTRAUX.

Résumé, par Ordre Alphabétique des Cours et Projéts de l'École Centrale, Augmenté de Tables Usuelles et d'un abrégé de Legislation. Par J. B., Ingenieur des Arts et Manufactures. Deuxième edition, revue, corrigée et complétée. Leather, 6 x 4 ins., 11 + 314 pp., illus. Paris, Vve. Ch. Dunod, 1904. 6 francs.

This handbook contains solutions of problems which engineers are obliged to solve offhand. These were worked out by the author in the courses at the École Centrale.

* Unless otherwise specified books in this list have been donated by the publishers.

They are presented in dictionary form and have been reduced to pocket size. The author has tried to exclude from the text all theories and long explanations in order to present the problems in a uniform and condensed manner. The manufacturer may find tables and formulas for verifying any plans and the engineer may find in convenient form the necessary matter for studying any project in mechanics, construction, electricity, hydraulics, etc.

BOILER CONSTRUCTION.

A Practical Explanation of the Best Modern Methods of Boiler Construction from the Laying Out of Sheets to the Completed Boiler. By Frank B. Kleinhans. Cloth, 8 x 6 ins., 421 pp., illus., 5 plates. New York, Derry-Collard Company, 1904. $3.00.

The preface states that the matter compiled for this work represents the most modern practice. In trying to get this matter in such shape as to be generally useful, the author has deemed it inadvisable to illustrate and describe the methods used by the builders of each of the many prominent boilers now being built for various classes of work; and, as the operations on different makes of boilers are so similar to each other, it has been considered best to devote a section to the description of boilers in general. Following this, the locomotive boiler is taken up in the order in which the work goes through the shop. Sections are given on testing boilers, and, finally, one section is devoted to useful tables. These tables have been grouped together, and are intended to give, as nearly as possible, all the data necessary in the construction of a boiler, together with the stresses which would be set up in the various members due to steam pressure and expansion. There is an alphabetical index of ten pages.

TREATMENT OF SEPTIC SEWAGE.

By George W. Rafter, M. Am. Soc. C. E. Cloth, 6 x 4 ins., 137 pp. New York, D. Van Nostrand Company, 1904. 50 cents.

In this book the author has attempted to give some of the more important developments in the bacterial treatment of sewage, but, owing to the limitations of size, none of these can be considered very complete. The work is, in effect, a series of hints, together with indications of preferable treatment. The author states that in such a book there is very little original work, and the facts and opinions are necessarily largely drawn from the studies of others. An attempt has been made, so far as possible within the limits of a book of this size, to place the septic method in its proper relation to other systems of sewage disposal.

TRAIN RULES AND TRAIN DISPATCHING.

A Practical Guide for Train Dispatchers, Enginemen, Trainmen and all who have to do with the Movement of Trains. By H. A. Dalby. Leather, 6 x 4 ins., 221 pp., illus. New York, Derry-Collard Company, 1904. $1.50.

The author states that this book is a series of suggestions which have come to him from time to time. In dealing with these subjects he has started at the beginning, hoping to be able to assist those who are in need of instruction. The author hopes that, with this book, the learner, with the experience he himself gains, may attain a valuable degree of proficiency. It is with this hope that this volume is offered as the result of a number of years of experience as operator and train dispatcher in various parts of the country. Some of the subjects dealt with are: Standard code for both single and double track; standard time at all points; clearance cards; staff system of running; protecting trains; special rules, train classification, rights, orders, signals and identification; work trains and extras, location of offices, etc. There is an index of four pages.

ELECTRICAL MOTOR INSTALLATIONS.

A Book for Factory Owners and Other Users of Steam Power. By F. J. A. Matthews, A. M. I. E. E. Boards, 9 x 6 ins., 194 pp., illus. Manchester, Scientific Publishing Company. 2 shillings 6 pence, net.

The author has prepared this second edition, in which a great portion of the matter in the first edition has been re-written and many additions made. He states that all technical terms have been avoided as far as possible. The chapter on motor installations in actual use has been extended, and one has been added on Electricity Applied to Colliery Work. There are thirty-one additional illustrations and an index of three pages.

Gifts have also been received from the following:

Abbot, Henry L. 1 pam.
Ala.—Geol. Surv. 2 bound vol., 3 vol., 6 pam.
Allen, Kenneth. 2 pam.
Am. Electrochemical Soc. 1 bound vol.
Am. Gas Light Assoc. 1 pam.
Am. Inst. of Elec. Engrs. 1 vol.
Am. Ry. Assoc. 1 pam.
Am. Soc. of Heating & Ventilating Engrs. 1 bound vol.
Anthony, Charles, Jr. 1 bound vol.
Architekten Verein zu Berlin. 2 pam.
Asociación de Ingenieros Industriales de Barcelona. 1 vol.
Associacao dos Engenheiros Civis Portuguezes. 1 vol.
Assoc. of Ry. Supts. of Bridges & Bldgs. 1 vol.
Augusta, Ga.—Supt. of Canals and Water-Works. 1 pam.
Australasian Inst. of Min. Engrs. 1 pam.
Baker, Ira O. 2 pam.
Baltimore, Md.—Water Board. 1 pam.
Bangor, Me.—Water Dept. 1 pam.
Baterden, James R. 1 pam.
Bay City, Mich.—Water Dept. 1 pam.
Bazin, M. H. 3 pam.
Binghamton, N. Y.—Board of Health. 1 pam.
Bituminous Coal Trade Assoc. 1 pam.
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Boston, Mass.—City Auditor. 1 bound vol.
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Escola Polytechnica do Rio de Janeiro. 1 vol.
Fendall, B. T. 1 pam.
Field Columbian Museum. 1 pam.
Fisk, W. L. 3 maps, 2 pam., 1 vol.
Forney, M. N. 1 pam.
France—Ministère des Travaux Publics. 2 vol.
Francis, George B. 2 pam.
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RESIGNATIONS.

	Date of Resignation.
Pratt, Robert James	May 31, 1904

DEATHS.

Barr, Jacob Neff	Elected Member, November 7th, 1888; died May 15th, 1904.
McCalla, Richard Calvin	Elected Member, December 5th, 1894; died June 13th, 1904.
McKeown, Thomas	Elected Member, December 3d, 1879; died June 7th, 1904.
Tingley, George Curtis	Elected Member, September 6th, 1871; died April 30th, 1904.
Tulloch, Alonzo J	Elected Member, June 6th, 1883; died July 21st, 1904.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(May 11th to August 3d, 1904.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City. 12c.
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Engineer*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, $1.
(29) *Journal*, Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des École Spéciales de Gand*, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Économique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill., 10c.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.
(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 410 Penn Ave., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(69) *Mill Owners*, New York City, 10c.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England
(72) *Street Railway Review*, Chicago, 30c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal, London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago. 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.

LIST OF ARTICLES.

Bridge.

The Substructure of Marsh River Bridge.* Herbert J. Wild, Jun. Am. Soc. C. E. (54) Vol. 52.
The Detailed Design of a Railroad Bridge. Wm. H. Burr and Myron S. Falk. (6) Apr.
Calculation of Stress from Actual Deflection. (21) May.
A Novel Piece of Bridge Construction at Lawrence, Mass.* M. F. Brown, Assoc. M. Am. Soc. C. E. (13) May 12.
A Concrete-Steel Highway Bridge at Plainwell, Mich.* P. A. Courtright. (13) May 12.
The Troitsky Bridge, St. Petersburg. (14) May 14.
Melan Concrete-Steel Arch Bridge across the Great Miami River, Dayton, Ohio.* (13) May 19.
A Pin Bridge that Stood without an End Post.* John C. Moses, Assoc. M. Am. Soc. C. E. (13) May 19.
Conway Suspension Bridge.* (12) May 20.
A Ballasted Railroad Bridge with a Plank Floor.* (14) May 21.
Types and Details of Bridge Construction; Plate Girders.* Frank W. Skinner. (14) May 21.
Ballasted Floor Through-Span Bridge for the Santa Fé.* (15) May 27.
The Substructure of the Fraser River Bridge.* (14) May 28.
Transporter Bridge across the Manchester Ship Canal.* (46) May 28.
Progress in Railroad Bridge Building. F. C. McMath. (Address delivered before the Detroit Engineering Soc.) (1) June.
The Landwasser Viaduct on the Albula Railroad.* (15) June 3.
The Elizabeth Bridge, Buda-Pest.* (19) Serial beginning June 4.
A French Breaking Test of Reinforced Concrete Bridge Trusses.* Leon S. Moisseiff, Assoc. M. Am. Soc. C. E. (14) June 4.
Articulations in Granite for Bridges.* (12) June 10.
The City Island Bridge, New York.* (14) June 11.
The Monocacy River Bridge.* (14) June 11.
The Eastern Oklahoma Railway Line and Bridges.* F. R. Patten. (40) June 17.
Railway Bridge over the Clyde at Uddingston.* (11) June 17.
A Short-Span Bascule Bridge in New York City.* (14) June 18.
A Three-Track Beam-Floor Girder Bridge.* (14) June 18.
The Mingo Bridge Approaches.* (14) Serial beginning June 25.
The Woodsville Railroad and Highway Bridge.* (14) June 25.
Arched Ribs. (21) Serial beginning July.
Architectural Features of the Manhattan Bridge.* (15) July 1.
The Connel Ferry Cantilever Bridge, Scotland.* (14) July 2.
The Manhattan Bridge Across the East River, New York City: Revised plans.* (13) July 7; (14) July 2; (46) July 9; (20) July 7.
The First Iron Railroad Bridge.* F. C. Coleman. (15) July 8.
The Broadway Bascule Bridge at Milwaukee, Wis.* John Geist. (13) July 14.
Some Observations of the Design of Reinforced Concrete Bridges. W. J. Douglass. (13) July 14.
The Paoli Trestle on the Monon.* A. E. Kemmer. (15) July 15.
The Reconstruction of Rye Swing Bridge.* E. R. Waight, Assoc. M. Inst. C. E. (12) Serial beginning July 15.
The St. Joseph Drawbridge of the Pere Marquette.* (15) July 22.
The Erection of a Three-Span Plate-Girder Bridge.* (14) July 23.
Replacing a Double-Track Drawbridge.* (14) July 30.
Concrete-Steel Bridges at Dayton, O.* (60) Aug.
Le Pont de Luxembourg.* R. Bonnin. (36) May 25.
Pont en Béton Armé, à Soissons.* Riboud. (33) June 18.
Reconstruction du Pont sur le Trent de la Compagnie du Midland Railway (Angleterre).* (33) July 9.

Electrical.

On the Calculation of Line Batteries.* W. E. Winship. (42) Apr.
Wave Form Variations of a Long-Distance Line.* George H. Rowe. (42) Apr.
Data and Tests on a 10 000 Cycle-per-Second Alternator.* B. G. Lamme. (42) Apr.
The Single-Phase Induction Motor.* William S. Franklin. (42) Apr.
On the Magnetic Dispersion in Induction Motors, and its Influence on the Design of These Machines. Hans Behn-Eschenburg. (77) Apr.
The Edison Accumulator for Automobiles. W. Hibbert, Assoc. M. Inst. E. E. (77) Apr.
Predetermination of Sparking in Direct-Current Machines.* W. L. Waters. (42) Apr.
Experiments on Eddy Currents.* W. M. Thornton, M. Inst. E. E. (77) May.
The Rated Speed of Electric Motors as Affecting the Type to be Employed.* H. M. Hobart, M. Inst. E. E. (77) May.
Three-Phase Working, With Special Reference to the Dublin System.* Wm. Brew, Assoc. M. Inst. E. E. (77) May.
The Central Electric Supply Co.'s Power Station.* (26) Serial beginning May 6.

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* Illustrated.

Electrical—(Continued).

Steam Alternator at Bedford.* (12) May 6.
Some Properties of Alternators under Various Conditions of Load.* A. F. T. Atchison. (Abstract of Paper read before the Inst. E. E.) (73) Serial beginning May 6.
The Engineering Laboratories at the South-Western Polytechnic, London. (47) May 7.
Power Station Design. C. H. Merz and Wm. McLellan. (Paper read before the Inst. E. E.) (26) May 13; (47) Serial beginning May 14.
Street Lighting by Nernst Lamps. Henry Joseph. (26) May 13.
The Conductometer. Rollo Appleyard. (26) Serial beginning May 13.
A Remote Control Switchboard for the London County Council Tramway Power House.* (73) May 13.
Automatic Protective Devices for Electrical Circuits.* (73) May 13.
Electrically-Registering Wind Vane and Anemometer for School Use.* H. W. Harmon. (19) May 14.
The Hewitt Mercury Vapour Lamp.* Magnus MacLean. (Paper read before the Inst. of Engrs. and Shipbuilders in Scotland.) (47) May 14.
Hudson River Power in Albany, Troy and Schenectady.* (27) May 14.
Boring Armature Bearings.* S. P. Baird. (72) May 20.
The Cable Steamer *Pacific*.* (73) May 20.
Losses in Cable Sheaths. M. B. Field. (Abstract of Paper read before the Inst. E. E.) (26) May 20.
Induction Coil Design.* Mortimer A. Codd. (26) Serial beginning May 20.
Manchester Traction, Light & Power Co.* (72) May 20.
Expansion of the Boston Edison System.* (27) May 21.
Some Aspects of New England Central Station Development. (27) May 21.
Résumé of Incandescent Electric Lamp Exhausting.* S. E. Doane. (27) May 21.
Measurement of the Insulation Resistance of an Electric Wiring System.* Edwin F. Northrup. (27) May 21.
The Protection of Telephone or Telegraph Lines when in Hazardous Proximity to High-Tension Lines.* Robert E. Chetwood, Jr. (27) May 21.
The Magnetite Arc Lamp.* Charles Proteus Steinmetz. (27) May 21.
Storage Battery Plate Construction with Special Reference to High Discharge Characteristics.* W. W. Donaldson. (27) May 21.
The Nernst Lamp.* (27) May 21.
A Self-Cohering Receiver.* Edward Van Winkle. (27) May 21.
General Electric Oil Switches.* (27) May 21.
Cape Town Corporation Electricity Works.* (73) Serial beginning May 27.
The Steam Turbine as Applied to Electrical Engineering.* Charles A. Parsons, G. Gerald Stoney and C. P. Martin. (Paper read before the Inst. E. E.) (73) Serial beginning May 27.
Electric Gear for Driving Bending Rolls.* (12) May 27.
The Repulsion Motor. F. W. Carter. (26) Serial beginning May 27.
The Largest Electric Water Power Station in New Hampshire.* (27) Serial beginning May 28.
An Expression for the Torque of a Polyphase Wattmeter.* F. R. Stowe. (27) May 28.
Alternating-Current Wave Form Analysis. S. M. Kintner. (27) May 28.
The Principles of the Repulsion Motor.* George T. Hanchett. (17) May 28.
Recording Alternating-Current Wave-Forms by Duddell Oscillographs.* (19) May 28.
The Development of Electric Power Transmission.* Lewis Buckley Stillwell. (10) June.
The Distribution of Electric Power from Transmission Systems. Louis Bell. (10) June.
Electric Power from Shawinigan Falls, Canada: Part I.—The Hydraulic Development.* Wallace C. Johnson; Part II.—The Electric Transmission Plant.* Ralph D. Mershon. (10) June.
Electric Power in European Collieries.* C. S. Vesey Brown, M. Inst. C. E. (10) June.
Electric Power in Manufacturing Plants.* Dugald C. and William B. Jackson. (10) June.
Electric Welding Development.* Elihu Thomson. (10) June.
Electricity and Light. Charles Proteus Steinmetz. (10) June.
Transformers for Long-Distance Power Transmission.* J. S. Peck. (10) June.
Limitations of Long-Distance Electric Power Transmission. Paul M. Lincoln. (10) June.
Metamorphosis of Wireless-Telegraph Apparatus.* A. Frederick Collins. (9) June.
Method of Making Electrical Survey of Rochester, N. Y.* (60) June.
Operation of Synchronous Converters. S. C. Lindsay. (42) June.
Some British Central Electric Power Stations.* H. F. Parshall, M. Inst. C. E. (10) June.
The Utilization of Niagara Power.* H. W. Buck. (Paper read before the Engineers' Soc. of Western New York.) (1) June.
A New Portable Capillary Electrometer.* L. Ramakers. (24) June 2.
Automatic Protective Devices for Electrical Circuits.* (26) Serial beginning June 3.
Conduits and Condensation. (26) June 3.
The Cost of Electric Energy.* G. L. Addenbrooke. (11) June 3.
The Dick-Kerr High-Tension Oil Switch.* (73) June 3.
A Ferraris Field Detector of Hertzian Waves.* Riccardo Arnò.* (73) June 3.
Kettering Electricity Works.* (26) June 3.

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* Illustrated.

Electrical—(Continued).

Efficiency Curves of Rotary Converters. A. S. McAllister. (27) June 4.
The New Electric Works at Neuchatel, Switzerland.* A. Steere. (14) June 4.
On a New Standard of Wave-Length.* James E. Ives. (27) June 4.
The Continuous Protection of Conductors.* A. Wireman. (26) June 10.
Why Use Carbon Brushes on the Modern Dynamo?* J. W. Burleigh. (26) June 10.
The New Holborn Telephone Exchange.* (73) June 10; (26) July 1.
Derby Electricity Works and Tramways.* (73) Serial beginning June 10.
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Hudson River Power in the General Electric Works.* (27) June 11.
The Davies-Perrett Electric System of Oil-Separation.* (11) June 17.
Faversham Electricity Works.* (73) June 17.
The Berthier Actino-Electric Transformer.* Emile Guarini. (46) June 18.
Exhibits of the Westinghouse Companies at the St. Louis World's Fair.* (18) June 18.
Influence of the Line on the Parallel Operation of Synchronous Motors.* (27) June 18.
Variation of Motor Speed With Variable Line Voltage.* J. L. Dickson. (27) June 18.
Storage Battery Plate Construction. S. H. Rabenalt. (27) June 18.
The Power Plants of the New Orleans Railways Company.* (17) June 18.
Practical Alternating Current Testing.* Chas. F. Smith, Assoc. M. Inst. C. E. (47) Serial beginning June 18.
The Missouri River Power Station of the Metropolitan Street Railway Company of Kansas City, Mo.* Howard Prescott Quick. (72) June 20.
The Commercial Testing of Sheet Steel for Electrical Purposes.* C. E. Skinner. (Paper read before the Amer. Soc. for Testing Materials.) (20) June 23.
Electrical Plant at Heysham Harbour.* (73) Serial beginning June 24.
Candle-Power Tests of Cooper Hewitt Mercury Vapor Lamp.* L. A. Freudenberger. (27) June 25.
Canadian Works of the Westinghouse Company.* (27) June 25.
Foucault Pendulum System.* By Ernest K. Adams. (27) June 25.
High Tension Transmission for Electric Railways.* W. J. Davis, Jr. (Paper read before the New England St. Ry. Club.) (17) June 25.
Lost and Unaccounted-for Current.* C. W. Humphrey. (Paper read before the National Elec. Light Assoc.) (24) Serial beginning July 4; Abstract (27) June 25.
Remodeled Studebaker Power Plant at South Bend, Indiana.* (14) June 25.
Some Details of an Electric Railway Power Station at Canton, Ohio. (14) June 25.
The Telefunken Ondometer for the Measurement of Wireless Telegraphy Waves.* (19) June 25.
The Edison Storage Battery.* (11) July 1.
The Induction Motor Diagram.* W. Parker. (26) July 1.
Photometric Tests of "Linolite."* (73) July 1.
Deltabeston Magnet Wire.* (17) July 2.
The Exhibit of the General Electric Company at the Louisiana Purchase Exposition.* (27) July 2.
Iron Losses in Loaded Transformers.* E. S. Johonnott. (27) July 2.
Kingsbridge Power Station of the New York City Railway Company.* (14) July 2; (17) July 2; (27) July 9.
The Large Switchboard at the St. Louis Exposition.* (27) July 2.
Light and Power Plant for the City of Geneva.* (19) July 2.
A Peruvian High-Voltage Transmission Plant.* A. L. Kenyon. (27) July 2.
Use of the Earth as a Return Conductor in Connection with Commercial Electrical Installations.* (19) July 2.
The New Two-Phase Station at Sheffield.* (73) July 8.
Organization of a Meter and Testing Department. A. J. Cridge. (Paper read before the Incorporated Mun. Elec. Assoc.) (73) July 8.
The Commercial Testing of Sheet Steel for Electrical Purposes.* (27) July 9; (47) July 9.
Electrical Equipment, C. and E. I. R. R. Shops at Danville, Ill.* (18) July 9.
New Hotel Plant in Denver. (27) July 9.
On the Parallel Working of Delta- and Star-Connected Three-Phase Transformers.* A. E. Kennelly and S. E. Whiting. (27) July 9.
The Electrical Equipment of Overhead Travelling Cranes.* J. W. Warr. (26) July 15.
The Neepsend Power Station of the Sheffield Electricity Department.* (26) Serial beginning July 15.
Polyphase Sub-Stations. S. L. Pearce. (Abstract of Paper read before the Incorporated Mun. Elec. Assoc.) (26) Serial beginning July 15; (73) Serial beginning July 15.
Power Plant at the St. Louis Exhibition.* (22) Serial beginning July 15.
The St. Louis Exhibition: Electrical Exhibits.* (11) Serial beginning July 15.
The Westinghouse Power-Plant at the St. Louis Exhibition.* (11) July 15.
Self Exciting and Compounded Alternators.* (73) July 15.
The Westinghouse Unit-Switch System of Multiple-Unit Train Control.* (73) July 15.
The Big Engine of the St. Louis Exposition and the Illumination of the Buildings.* (19) July 16.
The Inverted Repulsion Motor.* Karl Faber. (27) July 16.
The Rochefort System of Wireless Telegraphy.* A. Frederick Collins. (27) July 16.
New Telephone Exchange at Buda-Pest, Hungary.* Joseph Hollos. (27) July 16.
The Localisation of "Earths" on Feeders and Networks.* Horace Boot. (26) July 22.

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Electrical—(Continued).

Storage Batteries. (26) July 22.
Voltage Regulation in Alternating Current Systems. H. S. Meyer. (Paper read before the Liverpool Eng. Soc.) (11) Serial beginning July 22.
The Catawba River Power Development near Rock Hill, S. C.* C. A. Mees. (14) Serial beginning July 23.
A Hydro-Electric Power Development on the Catawba River, Near Rock Hill, S. C.* (27) July 23.
On the Complex Product of Electromotive Force, Current and Other Vectors. Henry T. Eddy. (27) July 23.
Motor-Driven Tools at the World's Fair, St. Louis.* (27) July 23.
Insulator Pins for Transmission Lines. Alton D. Adams. (10) Aug.
A Notable Mexican Hydro-Electric Plant.* Robert McF. Doble. (9) Aug.
Variable-Speed Motors. C. A. Seley. (25) Aug.
Note sur la Régulation des Groupes Électrogènes.* A. Neyret. (32) May.
Usine Hydro-Électrique de Kykkelsrud (Norvège).* (33) May 21.
Télégraphes Électriques pour la Transmission des Signaux à Bord des Navires.* L. Ramakers. (33) June 25.

Marine.

Ellis and Eaves' System of Induced Draught Applied to Marine Boilers.* (11) May 6.
Fire Prevention on Board Ship. Edwin O. Sachs. (Paper read before the Inst. of Naval Archts.) (11) May 6; (47) June 18.
Some Results of Model Experiments.* R. E. Froude. (Paper read before the Inst. of Naval Archts.) (12) May 6.
The Engines of H. M. Cruisers *Kent*, *Lancaster* and *Cornwall*.* (11) May 13.
Travelling Shipyard Crane at Vulcan Works, Bredow, Stettin.* (11) May 20.
The Heeling and Rolling of Ships.* A. Scribanti. (Paper read before the Inst. of Naval Archts.) (11) Serial beginning May 27.
The Problem of the Screw Propeller.* John Lowe. (19) May 28.
Warships with Six Propellers: Some Early Russian Types.* (10) July.
H. M. Torpedo-Boat Destroyer *Welland*.* (11) July 8.
Progress of Warships and Machinery Building in England. (12) July 8.
Water-Tight Subdivision of Warships. William Hovgaard. (Paper read before the Soc. of Naval Archts. and Marine Engrs.) (12) July 8.
The Submarine Torpedo Tube.* (46) July 9.
Methods of Estimating the Coal Endurance of a Naval Vessel.* D. W. Taylor. (13) July 14.
Launch of the Cunard Liner *Caronia*.* (12) July 15; (11) July 15.
The White Star Steamship *Baltic*.* (46) July 16; (15) July 15; (10) Aug.
Pumping Plant for a Floating Dock.* (12) July 22.
The New French Battleship *Democratie*.* C. Field. (46) July 23.

Mechanical.

The Newcomen Engine.* Henry Davey, M. I. Mech. E. (75) No. 1, 1903.
Experiments on the Efficiency of Centrifugal Pumps.* Thomas E. Stanton. (75) No. 3, 1903.
The Gas Stove Considered from a Chemical and Sanitary Standpoint. Francis C. Phillips. (58) Feb.
Analyses of Lubricating Greases. P. H. Conradson. (Paper read before the Engrs. Soc. of Western Pennsylvania, Chem. Section, and the Amer. Chem. Soc.) (58) Mar.
Gas Power. J. Emerson Dowson, Assoc. M. Inst. E. E. (77) Apr.
Modern High-Speed Steam Engines. (12) Serial beginning Apr. 15.
Wire Ropes.* G. W. Westgarth. (Abstract of Paper read before the National Assoc. of Colliery Managers.) (22) Apr. 29.
The Steam Turbine.* William Chilton. (77) May.
The Control of Furnace Combustion.* (11) May 6.
Some Early Machine Tools. (12) Serial beginning May 6.
Clearances of Reamer Cutters.* (47) May 7.
The Engineering Laboratories at the South-Western Polytechnic, London. (47) May 7.
Valves and Valve Mechanism of Internal-Combustion Engines.* Robert E. Phillips. (47) May 7.
Anti-Friction Alloys. John F. Buchanan. (47) May 7.
The Locke Steel Belt Machine for Automatically Manufacturing Steel Sprocket Chains.* (20) May 12.
The Burger Automatic Gas Engine.* (20) May 12.
Specifications for Machine Tools. (13) May 12.
Balanced Cable Cranes for Handling Excavated Material at Devonport, England, and Zambesi Falls, South Africa.* (13) May 12.
The De Laval Steam Turbine.* Charles Garrison. (From *Proceedings* of the Society of Arts.) (62) Serial beginning May 12.
Steam Turbine Discs. Maurice F. FitzGerald. (12) May 13.
The Thermal Effect and the Practical Utility of Superheated Steam. Robert H. Smith. (26) May 13.
Deschamp's Down Draft Gas Producer. (22) May 13.

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Westinghouse-Parsons Steam Turbine.* (27) May 14.
New Steam Turbine Development. (17) May 14.
Tests of Steam Turbines at the Newport Station of the Old Colony Street Railway. (14) May 14.
Motor Cars.* Alexander Govan. (Paper read before the Inst. of Engrs. and Shipbuilders in Scotland.) (47) Serial beginning May 14; (62) Serial beginning June 30.
The Gasoline Engine as Applied to Automobiles.* Albert L. Clough. (Lecture before the Boston Y. M. C. A. Automobile School.) (47) May 14.
A Few Notes on the Steam Turbine. G. L. Parsons. (Abstract of Paper read before the Inst. E. E.) (19) May 14.
Continuously-Propelled Automobile Trains.* Emile Guarini. (19) May 14.
Discussion of Published Data on the Thermal Efficiency of the Rotary Kiln, and Possible Reduction of Fuel Requirement. Henry E. Spackman, M. Am. Soc. M. E. (14) May 14.
Electrically-Driven Rolling Mills.* H. Koettgen. (Abstract of Paper read before the Verein Deutscher Eisenhüttenleute.) (20) May 19.
The Development of the Parsons Steam Turbine. (11) May 20.
Electrically-Driven Machines for Charging Gas Retorts.* E. Guarini. (26) May 20.
Soldering of Metals and the Preparation of Solders and Soldering Agents. (19) May 21.
Future Improvements in Internal Combustion Motors. (14) May 21.
Design for a 2-Cycle Gasoline Motor.* J. C. Brocksmith. (47) May 21.
An Application of Baffle Plates in a Feed-Water Heater.* (13) May 26.
The Rateau Steam Turbine.* (20) May 26.
Dynamic and Commercial Economy in Turbines. Robert H. Smith. (12) Serial beginning May 27.
Steam Curves.* W. H. Booth. (26) May 27.
Note on the Relative Efficiency of Heat-Insulating Media.* S. H. Davies. (12) May 27.
Elevating and Conveying Plant.* (47) May 28.
Influence of Casting Temperature on the Properties of Castings. Percy Longmuir. (47) May 28.
Conveying Belts in the Concrete Plant at the Washington Filters.* (14) May 28.
The Mathematics of Mufflers for Gasoline Engines. (19) May 28.
The Cyclograph.* Emile Guarini. (46) May 28.
The Casting of the Williamsburg Bridge Entablatures.* (46) May 28.
Two-Belt Conveyor System. (17) May 28; (27) June 11; (45) June.
Water Gas Tar Used in Generators for Enrichment. Henry I. Lea. (Paper read before the Ohio Gas Light Assoc.) (24) May 30.
Pipe Flanges. Gerald E. Flanagan. (64) June.
Centrifugal Fans. J. H. Kinealy. (70) Serial beginning June.
Motor-Driven Machine Tools.* (25) June.
Tube Ball Mills; Their Working and Mechanical Effects.* (67) June.
Cost of Building and Operating a Portland Cement Plant. Boilleau and Lyon. (60) June.
Modern Expanding and Flanging Machinery and Tools.* Luther D. Lovekin. (3) Serial beginning June.
Systems and Methods of Mechanical Refrigeration. Sterling H. Bunnell. (9) June.
Economizer Calculations. W. H. Booth. (64) June.
The Entropy Diagram and Its Uses.* H. Boyd Brydon. (64) June.
Expansion Curves.* (64) June.
Indicating the Gas Engine.* C. E. Sargent. (64) June.
Coal Gas and Water Gas; Advantages of Each and Cost of Manufacture. (45) June.
Efficiency of Rotary Cement Kilns.* W. Michaëlis, Jr. (67) June; (14) June 4.
Concrete Mixer with Automatic Measuring Device.* (13) June 2.
The Taylor-Newbold Metal Cutting Saw.* (15) June 3.
Hammer Cranes.* (22) June 3.
Recent Progress in Wire-Rod and Bar Rolling-Mill Plants. (11) June 3.
The Zoelly Steam Turbine.* (14) June 4; (27) June 11; (11) June 3; (12) June 3; (47) June 25; (62) June 30; (17) July 2.
Fly-Wheel Milling Machine.* (12) June 3.
The Air Compressor.* R. H. Collingham. (12) Serial beginning June 3.
Pielock Superheater System.* Alfred Gradenwitz. (46) June 4.
Encased Spring Pop Safety Valves.* (17) June 4.
Some Details of the Service Plant of the Louisiana Purchase Exposition.* (14) June 4.
Steam Pipe Coverings. S. H. Davies. (From *Journal* of the Soc. of Chem. Industry.) (47) June 4.
An American 100-Ton Breakdown Crane.* (12) June 10.
Crane Navvy.* (11) June 10.
A New System for Three-High Plate Mills.* (22) June 10.
A Factory Research Laboratory.* (12) Serial beginning June 10.
Petroleum and Its Use for Illumination, Lubricating and Fuel Purposes.* P. Dvorkovitz. (Abstract of Paper read before the Inst. Min. Engrs.) (22) June 10.
Brake Tests of a 400-kw. Westinghouse-Parsons Steam Turbine. (17) June 11.
Experiments Showing the Efficiency of Radiators for Gasoline Automobiles.* (19) June 11.
Power Plant of the Whitlock Branch of the American Cigar Company, Richmond.* (14) June 11.

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Mechanical—(Continued).

Distributing Gas at 25 Pounds Pressure Per Square Inch. George Helps. (24) June 18.
A New Balanced Automatic Trip Valve.* (20) June 16.
The Hainsworth Safety Catch for Elevators.* (20) June 16.
The Ridgway "Two-Belt" Conveyor.* (13) June 16.
Some Competitive Tests of Rock Drills for Air Consumption. (13) June 16.
Fuel Economy. George M. Carpenter. (15) June 17.
Analytical Valuation of Gas Coals. G. P. Lishman. (Paper read before the Inst. Min. Engrs.) (57) June 17; (24) June; Abstract (22) June 10.
Rateau Steam Turbine.* Walter Rappaport. (26) June 17.
Condensation Fallacies and Facts. W. H. Booth. (26) June 17.
Some Types of Modern Foundries.* (22) June 17.
Superheated Steam. F. J. Rowan. (Paper read before the Inst. of Engrs. and Shipbuilders.) (19) June 18.
The Arcade Building Power Plant, Dayton, O.* (14) June 18.
Purification and Combustion of Acetylene.* (From *La Nature.*) (19) June 18.
A Rotary Induction and Exhaust Valve for Explosion Engines.* (19) June 18.
The New Structural and Car Shops of the Cambria Steel Company, Johnstown, Pa.* (14) Serial beginning June 18
The Mechanical Stoker and the Human Operator. Edwin Yawger. (24) June 20.
The Missouri River Power Station of the Metropolitan Street Railway Company of Kansas City, Mo.* Howard Prescott Quick. (72) June 20.
Cupola Fan Practice.* (Abstract of Paper read before the Amer. Foundrymen's Assoc.) (20) June 23.
The Gleason 15-Inch Shearing Cut Bevel Gear Planer.* (20) June 23.
The Pioneer Charcoal Furnace and Chemical Plant. (20) June 23.
Home-Made Boiler Feed Water Heater.* Ernest Stein. (16) June 23.
Machine for Loading Wheelbarrows on Cars.* (13) June 23.
The Hutton Motor-Car.* (11) June 24.
5000-Horse-Power Reynolds Corliss Engine at the St. Louis Exhibition.* (11) June 24.
Stay-Bolt Iron. H. V. Wille. (Abstract of Paper read before the Amer. Soc. for Testing Materials.) (15) June 24; (47) July 16
A New Vertical Cross-Compound Engine.* (14) June 25; (17) June 18; (62) June 23; (72) June 20.
A Novel Four-Piece Mechanism.* (19) June 25.
Reversing Mechanism for Machine Tools.* (47) June 25.
Flow of Gas in Mains and Distribution at High Pressure.* W. C. Unwin, M. Inst. C. E. (Paper read before the Inst. of Gas Engrs.) (47) Serial beginning June 25.
Notes on the Steam Turbine. H. F. Schmidt. (17) June 25.
Gas Explosions. L. Bairstow. (24) June 27.
Balanced Cable Cranes for Handling Excavated Material.* (62) June 30.
The Flather Gear Cutter.* (20) June 30.
The Phillips Pressed Steel Pulley.* (20) June 30.
Mechanical Draft for the Boiler Plant at the St. Louis Exposition. (20) June 30.
The Becky-Brainard No. 1 14-inch Cutter and Reamer Grinder.* (20) June 30; (25) July.
The Gisholt Boring Mill.* (25) July.
Test of the Effect of Increasing Boiler Pressure on the Life of a Stay-Bolt. Milton J. Phillips. (Thesis Text, Sibley Col.) (25) July.
Industrial Locomotives for Mining, Factory and Allied Uses.* J. F. Gairns. (10) Serial beginning July.
Works Design as a Factor in Manufacturing Economy.* Henry Hess. (9) July.
The Choice of a Steam Plant, with Special Reference to American Electric Power Installation. George H. Barrus. (10) July.
Advanced Practice in Economical Metal Cutting.* Charles Day. (9) July.
The Grinding Machine as a Metal-Cutting Tool.* C. H. Norton. (9) July.
The Latest Cement Block Machine.* (60) July.
Dimension Limits and Limit Gauges; their Practical Uses and Results.* Arthur A. Fuller. (9) July.
The Tool Room and its Functions in Cost-Reduction.* John Ashford. (9) Serial beginning July.
Packing Machinery for Export. Paul Roux. (10) July.
Ferry Works, Queensferry.* (12) July 1.
Electric *vs.* Hydraulic Cranes for Riveter Towers. Frank B. Kleinhans. (27) July 2.
Standard Methods for Green Sand Beds.* Thos. D. West. (Paper read before the Amer. Foundrymen's Assoc.) (47) July 2.
A Machine for the Measurement of Screw-Threads.* (19) July 2.
The Oechelhauser Gas Engine.* (17) July 2.
The Pennsylvania Engineering Works.* (20) July 7.
The Edwards Conveyor.* (20) July 7.
Laying Submerged Pipe Lines at Buffalo, N. Y.* (13) July 7.
The Compressed Air Power Transmission Plant of the Cleveland Stone Company. Lucius I. Wightman. (13) July 7; (14) July 9.
The Specific Heat of Superheated Steam.* Robert H. Smith. (12) July 8.
Cooling Tower and Condensing Equipment in an Atlanta Plant.* (14) July 9.
A Proposed Universal Dictionary of Mechanical Drawing.* George H. Follows. (13) Serial beginning July 14.

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Theisen's Centrifugal Gas-Washer.* (11) July 15.
Boiler House Economies. Reginald S. Downe, M. Inst. E. E. (Abstract of Paper read before the Incorporated Mun. Elec. Assoc.) (26) July 15; (47) July 16.
An Apparatus for the Direct Determination of the Specific Gravity of Cement. Daniel D. Jackson. (Paper read before the Soc. of Chem. Industry.) (14) July 16.
Some Tests of Iron and Woodworking Machinery. C. H. Hines. (27) July 16.
Worm Gearing.* F. L. Berry. (47) July 16.
Test of a Westinghouse-Parsons Steam Turbine. (18) July 16.
Working Derrick of 100 Tons' Capacity, Pennsylvania R. R.* (18) July 16.
The Specific Heat of Superheated Steam.* (14) July 16.
Pumping and Air Compressing Machinery at the St. Louis Exposition.* (20) Serial beginning July 21.
A Riehle 600 000-Pound Testing Machine for the University of Illinois.* (20) July 21.
Carburetters.* J. S. V. Bickford. (12) July 22.
Racing Automobiles in the 1904 Gordon Bennett Cup Race.* (19) Serial beginning July 23.
Peat as a Fuel. J. Campbell Morrison. (Abstract of Paper read before the Robert Fulton Assoc. of the National Assoc. of Engrs.) (19) July 23.
Corrosion of Boiler Tubes. (From *Journal* of the Amer. Soc. of Naval Engrs.) (47) July 23.
Greasy Condensation Water as Boiler Feed.* William Paterson. (Paper read before the Junior Inst. of Engrs.) (47) Serial beginning July 23.
The Mechanical Plant of the Hotel Astor, New York City: The Power and Refrigerating Plant in the Latest Great Hotel.* (14) July 23.
Water-Hammer in Steam Pipes.* (14) July 23.
The Walker Tool Room Grinder No. 2.* (20) July 28.
Tod Rolling Mill Engines.* (20) July 28.
Modern Coal-Hoisting Apparatus.* Frank C. Perkins. (19) July 30.
Some Experiments on an Air-Cooled Petrol Motor. H. L. Callendar. (19) July 30.
Specialized Machine Tools.* Joseph Horner. (10) Aug.
A Test of a Motor Driven Planer.* J. C. Steen. (25) Aug.
On the Use of Bunsen Burners and Combustion Apparatus Without City Gas. H. D. Gibbs. (From *Journal* of the Amer. Chem. Soc.) (24) Aug. 1.
A New Electrical Process of Manufacturing Peat Fuel. (19) Aug. 6.
Le Carburateur Claudel, Précédé d'une Théorie Générale sur la Carburation.* H. Claudel. (32) Mar.
Étude sur la Production de la Vapeur.* A. Lencanchez. (32) Mar.
La Turbine à Vapeur du Système Rateau et Ses Applications.* J. Rey. (32) Apr.
Chargeurs Mécaniques pour Foyers de Chaudières à Vapeur.* L. Pierre-Guédon. (34) May.
Graisseur Mécanique à Clef-Distributrice: Système Lefebvre. (34) May.
Production et Applications du Froid Artificiel.* F. Cottarel. (33) Serial beginning May 21.
Laboratoire d'Essais du Conservatoire des Arts et Métiers.* A. Boyer-Guillon. (33) Serial beginning May 21.
Étude sur les Fours Rotatifs Appliqués à la Cuisson du Ciment.* J. Boèro. (36) Serial beginning May 25.
Sur la Réversibilité des Turbo-Machines Hydrauliques. Platon Yankowsky. (37) May 31.
Recherches sur les Dimensions à Donner aux Canaux de Distribution des Machines à Vapeur. F. Guthermuth. (Tr. by M. Lecuir.) (37) May 31.
Note sur le Moteur "Diesel."* L. Descans. (30) June.
Élévateur-Transporteur Électrique.* Joseph Costa. (33) June 11.
Le Laboratoire d'Essais du Conservatoire National des Arts et Métiers.* E. Leduc. (36) Serial beginning June 25.
Une Application Ingénieuse de la Commande Électrique des Outils.* Daniel Bellet. Serial beginning June 25.
Les Voitures a Moteur et Essieu Transformateur de Vitesse Henriod.* F. Doubrey. June 25
Les Bicyclettes; les Changements de Vitesse: Le Rétropédalage.* Carlo Bourlet. (33) Serial beginning June 25.
Machine Universelle d'Essais de 300 Tonnes du Laboratoire d'Essais du Conservatoire des Arts et Métiers.* Pierre Breuil. (33) July 2.
Nouveau Système de Transmission Pneumatique pour Lettres et Petits Colis.* (33) July 16.

Metallurgical.

The Smelting of Zinc Ores to Regain Spelter and Sulphuric Acid.* A. J. Diescher. (58) Feb.
Notes on Pyrometry.* M. E. J. Gheury. (Paper read before the Arc Works Eng. Soc.) (11) Serial beginning May 6.
The Kepp Metallurgical Furnace.* (20) May 19.
Amalgamation on the Rand. I. Roskelley. (From the *Journal* of the Chem Metal. and Min. Soc. of S. Africa.) (16) May 26.
Cyaniding Gold-Bearing Sulphurets. S. B. Christy. (Paper read before the California Miner's Assoc.) (48) June.
Pyrite Smelting: A Review. Edward D. Peters. (16) Serial beginning June 2.
Mesabi Ores in Coke Blast Furnace Practice. W. A. Barrows, Jr. (62) June 9.

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Metallurgical—(Continued).

Furnace Top Explosions. Frank C. Roberts. (62) June 9.
The Keller Electrical Steel Process. (From *Eisen-Zeitung.*) (20) June 9; (62) July 14.
The Burgers System of Blast Furnace Construction.* (20) June 9.
The Ball Engine Company's New England Shop.* (20) June 9.
The Use of Thermite in Producing Pure Metals and Alloys. (19) June 11.
Electrolytic Iron. C. F. Burgess and Carl Hambuechen. (Paper read before the Amer. Electro-Chem. Soc.) (47) June 11; (73) June 17.
The Preparation of Brown Hematite Iron Ores.* F. Lynwood Garrison. (16) June 16.
Mesaba Fine Ore and Clinkered Ore. A. D. Elbers. (62) June 16.
A New Process of Galvanizing. (12) June 17.
A Furnace Charging and Distributing Apparatus.* Frank C. Roberts. (20) June 23.
Alloy Steels. William Metcalf. (Paper read before the Amer. Soc. for Testing Materials.) (20) June 23; Abstract (15) June 24; (14) July 2.
Magnetic Concentration of Zinc Ore in Virginia.* C. Q. Payne. (16) June 23.
Nitrogen in Iron and Steel. Ernest A. Sostedt. (62) June 23.
Smelting Iron by Electricity. (62) June 23.
Notes on Processes for Producing Open-Hearth Steel.* R. M. Daelen. (22) June 24.
A Blast Furnace of the Latest Type. (16) June 30.
Ore Dressing at Cananea.* Dwight E. Woodbridge. (16) June 30.
Direct Casting from the Blast Furnace. (12) July 1.
Treatment of Complex Ores. (62) July 7.
Electric Steel Furnace at Gysinge, Sweden.* F. A. Kjellin. (62) June 2; (12) July 8.
Some Notes on the Magnetic Separation of Ore.* (22) July 15.
The "Spooks" of the Blast Furnace. A. D. Elbers. (62) July 21.
The Care of Cyanide Solutions. W. H. Davis. (16) July 21.
Investigations of the Blowing of Foundry Cupolas. (13) July 21.
Steel Making at Ensley, Alabama.* Day Allen Willey. (19) July 23.
The Manufacture of Iron by Electro-Metallurgical Processes.* Adolphe Minet. (9) Aug.
The Electro-Metallurgy of Iron and Steel. Émile Guarini. (19) Aug. 6.
Recherches sur les Aciers au Tungstène. Léon Guillet. (33) Serial beginning May 7.
Électrométallurgie. L. François et L. Tissier. (36) Serial beginning May 10.
Fabrication d'Aciers au Nickel avec une Pyrite Magnétique Cupronickelifère.* (33) July 2.
La Fabrication des Plaques de Blindage aux Forges Nationales de la Chaussade.† A. Bizot. (33) July 23.

Military.

Creusot and the Ordnance Made There.* L. Ramakers. (19) June 18.
Japanese Naval Guns.* (46) July 2.
Mechanically-Propelled Vehicles for Military Purposes. (11) July 8.
Automatic Rifles.* (11) July 22.

Mining.

Freezing as an Aid to Excavation in Unstable Material.* James H. Brace, Assoc. M. Am. Soc. C. E. (54) Vol. 52.
Proposed Method of Sinking Through Soft Surface. James R. Wilson. (59) Vol. XXVI, Pt. 1.
Suggested Amendments to the Proposed New Regulations for the Use of Electricity in Mines. (22) Apr. 29.
Washing Plant at the Great Western Colliery.* (22) May 6.
The Application of Coal-Cutting Machines to Deep Mining.* W. E. Garforth, M. Inst. C. E. (Paper read before the National Assoc. of Colliery Managers.) (22) May 6.
The Coal and Iron Resources of the Dominion of Canada.* (22) May 6.
A Gravity Incline at the Antonio Mine in Cuba.* E. M. Holmes. (13) May 12.
Electric Winding. Gerald Hooghwinkel. (22) May 13.
British and American Coal-Cutting Machines. A. S. E. Ackermann. (Abstract of Paper read before the Soc. of Engrs.) (68) Serial beginning May 7; (22) May 13.
Grit Mills *v.* Wheeler Pans. O. B. Ward. (68) May 14.
The Hancock Jig. F. Danvers Power. (16) May 19.
Electrically Driven Hauling and Winding Gears.* (22) May 20.
Recent Electrical Developments (in mining machinery).* G. H. Hollingworth. (Paper read before the National Assoc. of Colliery Managers.) (22) May 20.
Methods of Working Coal in the Rhenish Westphalian Basin.* Maurice Bodart. (Abstract from *Report* in "Annuaire de l'Association des Ingénieurs Sortis de l'Ecole de Liége," Vol. XVII.) (22) May 27.
Electric Power in European Collieries.* C. S. Vesey Brown, M. Inst. C. E. (10) June.
The Primero Mines.* R. M. Hosea. (45) June.
Ropes of Vegetable Fiber (used in mining).* Charles W. Comstock. (45) June.
A Comparison of Three-Phase and Continuous Currents for Mining Purposes. Roslyn Holiday. (Abstract of Paper read before the Inst. Min. Engrs.) (22) June 3.
Electrically-Operated *Versus* Steam-Driven Mine Hoists.* (46) June 4.
A Gold-Mining Dredge of Recent Design * Samuel S. Wyer. (16) June 9.
Safety Lamps and Explosives: Experiments in Belgium. V. Watteyne. (Paper read before the Inst. Min. Engrs.) (57) June 10; Abstract (22) June 10.

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Prevention of Accidents in Winding.* J. H. Merivale. (Paper read before the Inst Min. Engrs.) (57) June 17; Abstract (22) June 17; (26) June 17.
Electric and Compressed Air Locomotives for Mines. Beverley S. Randolph. (Paper read before the Inst. Min. Engrs.) (57) June 17; Abstract (22) June 8.
Electric Rock Drills.* Gerald Hooghwinkel. (22) June 17.
Electricity in Colliery Work.* F. J. A. Matthews, Assoc. M. Inst. E. E. (47) Serial beginning June 18.
Underground Temperatures, Especially in Coal Mines. Hans Hoefer. (Abstract of Paper read before the Inst. Min. Engrs.) (68) Serial beginning June 18.
Electrically-Driven Mining Fan.* (12) June 24.
The Colliery Exhibition at the Royal Agricultural Hall. London.* (22) Serial beginning June 24; (57) July 1; (26) Serial beginning July 1; (73) July 8.
Coal Cutting by Machinery.* G. L. Kerr. (22) June 24.
Electrical Mining Machinery.* (12) June 24.
The Siemens-Ilgner System for Electric Mine Hoists.* (16) June 30.
Industrial Locomotives for Mining, Factory and Allied Uses.* J. F. Gairns. (10) Serial beginning July.
Hoisting-Engine Brakes.* J. S. Lane. (45) July.
Hoisting Engine Foundations.* R. V. Norris. (45) July.
Hoisting Engine Specifications. F. W. Gerecke. (45) July.
Compounding in Hoisting Engines. (45) July.
Deep Hoisting in South Africa.* J. S. Lane. (45) July.
Types of Hoisting Engines.* S. T. Nicholson. (45) July.
Care and Handling of Hoisting Engines. J. H. Pennington. (45) July.
Cylindrical *vs.* Conical Drums.* F. Moeller. (45) July.
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Tail- or Balance-Rope Hoists in Shafts in the Anthracite Regions of Pennsylvania.* Arthur H. Storrs. (45) July.
Deep Hoisting in the Lake Superior District.* O. P. Hood. (45) July.
Compressed Air in Hoisting.* J. S. Lane. (45) July.
Drums *vs.* Reels for Hoisting.* F. F. Coleman. (45) July.
Hoisting-Engine Calculations. (45) July.
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Balanced and Unbalanced Hoists.* E. T. Sederholm. (45) July.
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Blasting Tight Placers Before Dredging.* Oliver B. Finn. (16) July 7.
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Steel to Replace Timbering.* R. V. Norris. (16) July 14.
Kerosene Shale in Australia.* John Plummer. (16) July 14.
Head-Frames and Tipples.* W. R. Crane. (16) July 14.
New Water Spraying Apparatus for Dusty Mines.* (22) July 15.
Annual Report of His Majesty's Inspectors of Explosives, 1908. (57) July 15.
Coal Winding Machinery.* Horace D. B. How. (22) July 8; (47) July 28.
Marble Quarrying in America: A New Field for Electric Power Service.* Day Allen Willey. (10) Aug.
Mineral Shipments at British Ports.* Brysson Cunningham, Assoc. M. Inst. C. E. (10) Aug.
Effets du Froid et de la Gelée sur les Explosifs Industriels en Usage dans les Exploitations Minières.* H. Schmerber. (33) May 7.
Installation de Deux Sièges d'Extraction aux Mines de Fer d'Ely, Minnesota (États-Unis).* (33) May 14.
De la Transmission et de l'Utilisation de la Force dans les Mines.* Hector Denis. (34) Serial beginning June.
Appareils de Sécurité en Usage dans les Exploitations Minières d'Autriche-Hongrie.* H. Schmerber. (33) Serial beginning June 11.
La Lutte contre les Incendies et la Protection des Pompiers contre le Feu et l'Asphyxie.* (36) July 10.

Miscellaneous.

Transatlantic Engineering Schools and Engineering. R. Mullineux Walmsley. (77) May.
Methods for Determining the Equations of Experimental Curves.* A. S. Langsdorf. (Paper read before the Engineers' Club of St. Louis.) (1) June.
Methods and Cost of Sodding. Arthur Hay. (13) June 2.
Organizing a Civil Engineering Corps. F. H. Newell, M. Am. Soc. C. E. (13) July 21.

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Street Lighting by Nernst Lamps. Henry Joseph. (26) May 13.
Neglected Details in Brick Pavement Construction. Ira O. Baker. (76) June.
Fire Protection for Factories: How Its Value Shall Be Determined and Who Shall Pay for It. J. H. Purdy. (28) June.

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Progress of Plans and Work for Rebuilding Baltimore.* Daniel J. Hauer. (13) June 2.
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Effect of Heat upon Asphaltum in Paving Mixtures. F. O. Blake. (60) July.
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City Gas Inspection and Inspectors. Ira C. Copley. (Paper read before the Congress of Gas Associations of America.) (24) Serial beginning Aug. 1.
Le Goudronnage des Routes.* G. Espitallier. (33) May 14.

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The Location of the Knoxville, La Follette and Jellico Railroad, of the Louisville and Nashville System.* W. D. Taylor, M. Am. Soc. C. E. (54) Vol. 52.
Method Used by the Railroad Commission, of Texas, under the Stock and Bond Law, in Valuing Railroad Properties. R. A. Thompson, Assoc. M. Am. Soc. C. E. (54) Vol. 52.
Automatic Couplers.* Frederick C. Hibberd, M. I. Mech. E. (75) No. 6, 1903.
Electric Traction with Alternating Currents.* A. C. Eborall, M. Inst. E. E. (77) Apr.
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Six Coupled Tank Engine: Great Western Railway.* (12) May 13.
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Some Modern Appliances Connected with Railway Crossings and Points.* Owen R. Williams. (Abstract of Paper read before the Inst. of Engrs. and Shipbuilders in Scotland.) (22) May 13.
Higher Train Pipe Pressure for Passenger Service. J. P. Kelly. (Abstract of Paper read before the Air-Brake Assoc.) (15) May 13.
Brake-Beams and Foundation Brake Gear. T. A. Hedendahl. (Abstract of Paper read before the Air-Brake Assoc.) (15) May 13; (40) May 13.
Railroad Fusees. J. C. Ramage and Lewis Cheeseman. (15) May 13.
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Handling Trains with Back-Up Hose.* (Report of Committee of Air-Brake Assoc.) (40) May 13.
A Novel Block or Telephone Signal System. (19) May 14.
The Alternating Track Circuit System of Signaling. J. B. Struble. (Paper read before the Railway Signal Assoc.) (40) May 20; (18) May 21; (13) May 26; (15) May 20.
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Compressed Air and Pneumatic Tools in Railroad Service. Thos. Aldern. (Abstract of Paper presented to the Central Ry. Club.) (15) May 20.
A Rational Basis for Calculating the Braking Power of Passenger Cars. (15) May 20.
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Train Testing. Sydney W. Ashe. (17) May 21.
Railroading Across Great Salt Lake.* Herbert I. Bennett. (19) May 21.
A Turntable of New Design.* (18) May 21.
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Shapes of Ties.* Herman von Schrenk. (From Bulletin No. 50, U. S. Dept. of Agriculture) (15) June 24; (17) June 11.
Shop Notes from Elizabethport.* (15) June 24.
Stake Pockets.* (Report of Committee to the Master Car Builders' Assoc.) (15) June 24; (18) July 9.
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The Use of Modern Telephones as Applied to Railroads. B. A. Kiser. (Paper read before the Assoc. of Railway Telegraph Sup'ts.) (40) June 24.
To Prevent Rust on Steel Cars. (Report of Com. to the Master Car Builders' Assoc.) (15) June 24; (39) July; (18) July 9; (62) July 21; Abstract (73) July 16.
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Steel in Passenger Car Construction.* William Forsyth, M. Am. Soc. M. E. (Paper read before the Master Car Builders' Assoc.) (39) July; Abstract (15) June 24; (18) June 25; (13) June 30.
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Les Nouvelles Locomotives 51-65 du Chemin de Fer de la Petite Ceinture de Paris. J. Koechlin. (38) May.
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Method of Making Electrical Survey of Rochester, N. Y.* (60) June.
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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

GENERAL METHODS FOR THE CALCULATION OF STATICALLY INDETERMINATE BRIDGES, AS USED IN THE CHECK CALCULATIONS OF DESIGNS FOR THE MANHATTAN BRIDGE AND THE BLACKWELL'S ISLAND BRIDGE, NEW YORK.

By Frank H. Cilley, S. B.

To be Presented September 21st, 1904.

Two huge suspension bridges, the old "Brooklyn Bridge" and the "Williamsburg Bridge," now span the East River, uniting Manhattan with the adjacent boroughs of Greater New York on Long Island. Two other great bridges across the East River have been projected, and are now in process of construction, the "Blackwell's Island Bridge" and the "Manhattan Bridge," designed by the City Department of Bridges, under the direction of Gustav Lindenthal, M. Am. Soc. C. E., former Commissioner of Bridges, assisted by a consulting architect, Mr. Henry F. Hornbostel. Subjected to the criticism,

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

advice and approval of the Municipal Art Commission, a body of eminent architects and artists, and of a special engineering commission of five of the most prominent bridge engineers of the country, they promise to give New York two of the most handsome and serviceable great bridges of the world. And as the same broad policy, of giving the city the benefit of the advice, criticism and expert knowledge of specialists outside of the department, is being pursued in matters of detail, as in drawing up the specifications and making independent or check sets of stress calculations, similar freedom from fault and error may be expected here.

The methods used in making the check sets of stress calculations, which were assigned to, and have been made by, the writer, is the subject of this paper. It is presented in the belief that the methods described are so exceptionally simple, so easy of application, so general, as well as so accurate and reliable, that they are well worth the attention of American bridge engineers generally, and with the hope that the exceptional magnitude of the two great bridges to which these methods were applied with perfect success, will win for them attention and consideration which otherwise could hardly be secured. These methods include nothing new; that is, nothing not already well known and used in Europe; but, as far as the writer is aware, they are little known in this country and have not previously been described in a complete, brief and clear manner. It is the purpose of the writer to attempt such a description, and he trusts that the reader will pardon any repetition of matter already well known, the inclusion of which is made necessary for the sake of completeness.

The two bridges in question are what are known as "statically-indeterminate structures," *viz.*, structures with more inner parts or outer restraints than are necessary for their definition (or to make them "just stiff"), and the calculation of the stresses, therefore, is not possible through the simple application of the equations of equilibrium. For this calculation there is required, in addition, the consideration of distortion under load and the consequent distribution of stress among the different parts of the structure. This involves, in turn, knowledge, not merely of the form of the structure, but of the dimensions, material and elastic qualities of its parts, and necessitates much longer and more complicated calculations, the results of which are much less reliable* than in the case of the simpler statically-determinate structures. However, these calculations are frequently necessary, as in this case, and we are wholly concerned here with the exposition of a simple, clear and reliable general method for this purpose.

If the structure were determinate, *i. e.*, if it had no unnecessary outer restraints or inner parts, the calculation of the stresses by the ordinary laws of equilibrium would offer no difficulties. To make any actual indeterminate structure so calculable it would be necessary to remove the superfluous restraints and cut the superfluous parts. But in that case the structure would no longer act as it did previously. Suppose, however, that the restraints thus removed and the stresses at the cut surfaces thus eliminated, be replaced by forces such as will make the distortion of the structure exactly what it would have been if the superfluous restraints had not been removed and the superfluous parts had not been cut. The stresses would then be maintained as though nothing had been changed, and they would readily be expressed in terms of the loads and the replacing forces. But these replacing forces are unknown. How shall they be determined? Simply by requiring them to be such that the distortions which they, together with the given loading, cause, shall be precisely the same throughout as though no restraints had been removed and no parts had been cut. That is, they must be such that the points free from restraint will, nevertheless, follow the path of restraint, and the cut surfaces of the parts cut shall, nevertheless, remain together as though they were not cut. These conditions we may express through equations of elastic distortion, and these equations will be found precisely equal in number to the unknown forces, and linear in form. Let us consider the particular cases of the Blackwell's Island and the Manhattan Bridges.

Both these bridges are frameworks, and are treated under the simplifying assumptions usual with frameworks, the calculation being limited, in both cases, to the main truss systems.

In the case of the Manhattan Suspension Bridge we have a structure of three spans to consider, *viz.*, one center span of 1 470 ft. and two side spans of 725 ft. each. The center span is an inverted

* See "Some Fundamental Propositions Relating to the Design of Frameworks," *Technology Quarterly*, June, 1897; "Some Fundamental Propositions in the Theory of Elasticity,—a Study of Primary or Self-Balancing Stresses," *American Journal of Science*, April, 1901, and "The Exact Design of Statically Indeterminate Frameworks, An Exposition of its Possibility, but Futility," *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 353.

arch supported on rocking towers. The side spans are inverted semi-arches, each supported by a tower at one end and an anchorage at the other. Horizontal motion of the anchor ends of the anchor spans is prevented by anchor chains, which are continuations of the top chords and which extend far down into the anchorage masonry, where they are made fast.*

Suppose one of the anchor chains cut; the structure would be reduced to three simple spans and become at once statically determinable. But it would not act as before. The cut anchor chain stress must be replaced by a pair of equivalent forces, applied to the cut ends. In order to determine this pair of forces, it is only necessary to express the condition that the cut ends of the chain shall remain just touching, as a result of the distortion of the structure from these forces and the load. Or, suppose that the middle lower chord bar of the center span be cut, that a hinge be introduced in the middle of the upper chord bar above it, and that suitable diagonals to this hinge be provided, in place of the normal pair of counters of the center panel. The structure is then reduced to an inverted three-hinged arch on two rocking piers braced by two anchored side spans, and, as before, becomes statically determined. Again, it would not act as before, and the stress in the cut bottom chord bar must be replaced by a pair of equal and opposite forces applied at the cut ends. In order to determine these forces it is only necessary to express the condition that the cut ends of the bar shall remain just touching, as a result of the distortion of the structure from these forces and the load.

In the case of the Blackwell's Island Bridge we have a cantilever structure of five clear spans to consider, *viz.:* (1) a Manhattan shore span of 469.5 ft.; (2) a cantilever arm of 591 ft. extending over the west channel of the East River, met by a similar cantilever arm of 591 ft. from the Blackwell's Island truss, making a river span of 1 182 ft.; (3) the main span of the Blackwell's Island truss over the island, 630 ft.; (4) a cantilever arm of 492 ft. over the east channel of the East River, met by a similar cantilever arm of 492 ft. from the Queens

* The analysis of this structure as to the degree of indetermination of the outer forces may be made as follows: There are four points of support for the truss system; therefore, with two reaction elements at each support, there would be eight reaction elements to determine; but at the towers the reactions are vertical, thus leaving only six elements to be determined. For this, statics furnishes the three conditions of the equilibrium of forces in a plane and the two further conditions that the moment of all outer forces either side of a section at either tower, about the summit of that tower, is zero. One condition remains to be furnished through elastic relations.

truss, making another river span of 984 ft.; and (5) the shore span of the Queens truss, of 459 ft. Unlike usual American cantilever bridges, which are of the statically determinate type, the ends of the cantilever arms are not here separated, horizontally, by a gap spanned by a connecting truss, but the end of the cantilever arm from one shore comes vertically over the end from the other shore, and the two are connected by a long, rigid, vertical member or "rocker arm." These two rocker arms, one in each river span, make the structure doubly statically indeterminate.*

Suppose both these rocker arms cut, the structure would be reduced to three simple spans with disconnected cantilever extensions and would become statically determinable. To make it act as before, the rocker arm stresses must be replaced by pairs of equal and opposite (unknown) forces applied to the cut ends. In order to determine these forces we must express the conditions that the cut ends of the bars shall remain just touching, as a result of the distortion of the structure from these forces and the load, two conditions (since there are two cut bars) corresponding to the two pairs of forces which are to be determined.

In all cases, then, the problem is reduced to that of expressing certain displacements of the structure in terms of a set of forces, some known and some unknown, acting upon it as a statically determinate structure. We may, therefore, by the usual methods, and simply through the laws of equilibrium, express the stresses in all parts of the structure in terms of these forces, known and unknown, and thence, through the stress-strain relations for the given materials and for the given sections, express the strains. It only remains to express the displacements sought in terms of these strains, equate them properly and obtain the required conditions. Thus far, probably nothing has been stated that is not familiar to all. But how shall we express the desired displacements in terms of the strains of the parts?

Various methods may be adopted to this end. Until quite recently,

* The analysis of this structure as to the degree of indetermination of the outer forces may be made as follows: There are six points of support of the truss system; therefore, with two reaction elements at each support, there would be twelve reaction elements to determine; but each shore span anchorage reaction is vertical and one of the Blackwell's Island truss reactions is also vertical, thus leaving only nine elements to be determined. For this, statics furnishes the three conditions of the equilibrium of forces in a plane, the two conditions that the moment of all outer forces on either side of a section through either rocker arm about any point of the arm is zero, and the two further conditions that the resultant horizontal component of all outer forces on either side of a section through either rocker arm is zero. Two conditions remain to be furnished through elastic relations.

methods were applied which were either accurate, but very laborious, or, less laborious, but only approximate, often based on theorems of flexure. But, not much more than twenty or thirty years ago, it was noted that the "principle of virtual velocities" could be applied to this end. Briefly stated, this theorem simply asserts that the displacement of a point, A, of an elastic body in a given direction, resulting from a given displacement or distortion of some other part of the body, is equal to the work done at that other part of the body by the stresses there due to unity force at A in the direction of its displacement, through their displacements. Specifically, if a bar, subject only to direct stress, be extended by Δ, and unity force applied at A in the direction in which its (A's) displacement was desired caused a stress, S', in the bar, then $S'\Delta$ is the displacement of A in the given direction caused by the extension, Δ. The basis of this statement is very simple. The work done by the force unity at A, through its displacement, must equal the corresponding work done by the stress, S', through the stretch of the bar, since these are forces of a system in equilibrium, and the only ones whose relative displacement is involved. This means, of course, that the structure must be such that the independent extension of the bar is possible; in a word, it must be statically determined to that extent. Thus, to get the displacement of A in a given direction we have only to apply at A an imaginary force of unity in that direction, determine the corresponding stresses in the various parts of the (artificially) statically determined structure, find the work (positive or negative) done by these through the actual strains, find the algebraic sum of these work elements, and the result will be the actual displacement of the point, A, in the given direction, as a result of the given set of strains. Analytically, this is expressed for a framework in the well-known formula, $\delta = \Sigma\, S'\,\Delta$, in which S' expresses the stresses in the bars due to unity force at A in the direction of its required displacement, δ, and Δ expresses the actual elongations of the corresponding bars from whatever cause, as stress, temperature change, etc. The elongations, Δ, due to stress are, for prismatic bars of perfectly elastic material, of course,

$$\Delta = \frac{S\,l}{E\,a};$$

and the stresses, S, may be expressed as simple linear functions of the loads, L, and the unknown forces replacing the removed restraints and stresses. Ultimately, this evidently results in the expression of each δ (and therefore of each of the required conditions), in terms of a simple linear equation of the forces known and unknown. The solution of these linear equations, which are equal in number to the unknown forces, gives their values.*

Turn again to the particular examples to be considered. Suppose, in the Manhattan Bridge, that we cut an anchor chain and that we made the horizontal component of the stress in the anchor chain, H, the unknown force to be determined. Suppose we cut the Manhattan anchor chain at the shore end of the Manhattan shore span. Let us find the analytic condition that under any given load and the corresponding H, the separation of the cut ends of the anchor chain shall be zero. Or, as a still better expression, in this case, let us find the separation of the cut ends of the anchor chain due to any given load, and the horizontal pull, H, necessary to reduce this to zero. This H will then be the required value.

First, let us find how much separation of the cut ends a horizontal pull of unity would overcome. Let S' be the stresses throughout the structure resulting from the application of a pair of equal and opposite pulls whose horizontal component is unity, on the cut ends of the cable—that is, the stresses in the structure due to unity "horizontal component" (see Fig. 1). The elongations of the bars under these stresses would be,

$$\varDelta_{H=1} = \frac{S' l}{E a};$$

and the separation which would be overcome by $H = 1$, would be,

$$\delta_{H=1} = \Sigma\, S' \varDelta_{H=1} = \Sigma \frac{(S')^2\, l}{E a}.$$

What would be the separation of the cut ends of the anchor chain due to a given load acting alone? What, for example, would be the separation due to a load of unity applied at the center of the center span? Let S'' be the stresses throughout the structure due to unity load at that point. It must be remembered, here, that we are dealing with the structure with an anchor chain cut, and therefore acting as three simple spans. The S'' are simply the stresses in the middle span and the towers, due to load unity at the center of the middle span acting

* See George F. Swain—"On the Application of the Principle of Virtual Velocities to the Determination of the Deflections and Stresses of Frames." *Journal* of the Franklin Institute, Feb., 1883.

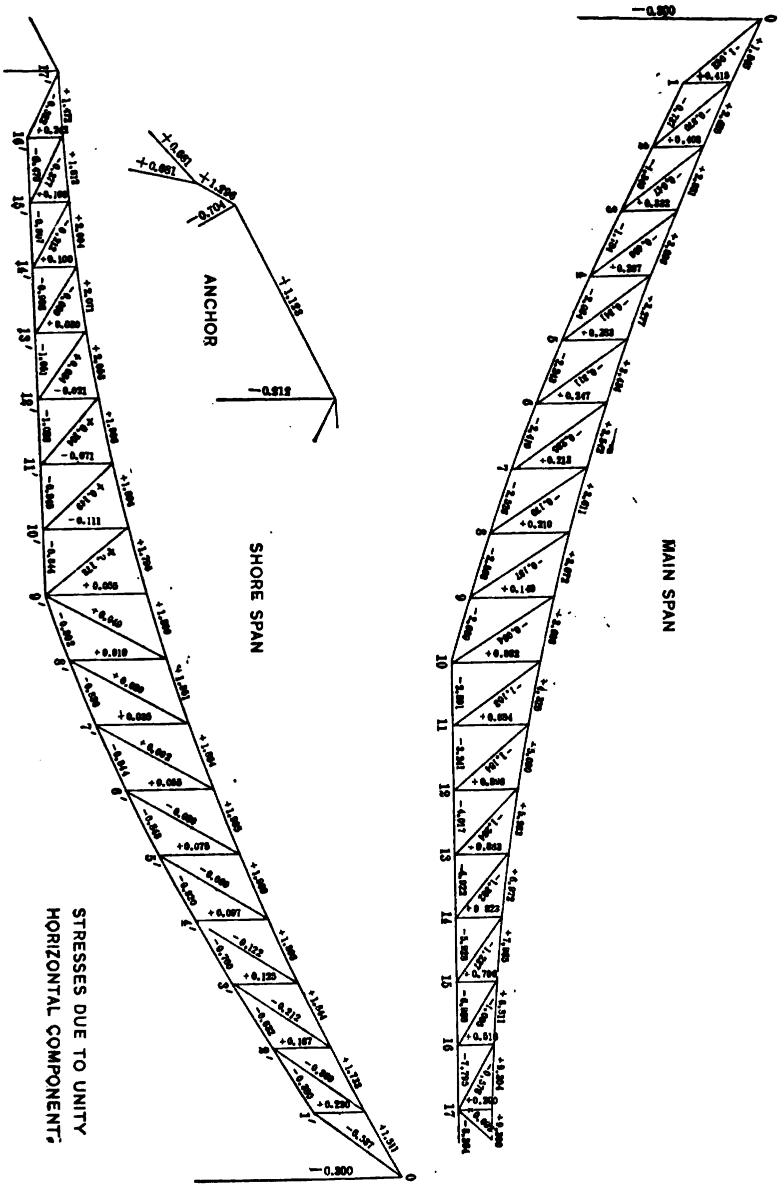

ANCHOR
SHORE SPAN
MAIN SPAN
STRESSES DUE TO UNITY
HORIZONTAL COMPONENT.

Fig. 1.

as a simple truss supported at its ends. The consequent elongations of the bars are,

$$\Delta'' = \frac{S'' l}{E a},$$

and the separation of the cut ends of the anchor chain resulting from these elongations is,

$$\delta'' = -\Sigma S' \Delta'' = -\Sigma S' \frac{S'' l}{E a},$$

the minus sign being used because the S' are due to unity pull, that is, they are forces acting contrary to separation. And since unity horizontal component will overcome $\delta_{H=1}$ separation, the actual horizontal component due to a load of unity at the center of the center span of the actual structure (with uncut anchor chains) must be,

$$H'' = \frac{\delta''}{\delta_{H=1}} = -\frac{\Sigma S' \frac{S'' l}{E a}}{\Sigma \frac{(S')^2 l}{E a}}.$$

In the same way, by applying a load of unity at any other point, determining the consequent stresses and thence the strains of the members of the structure (with anchor chain cut), and so the consequent separation, δ, of the cut ends of the anchor chain, we could find the value of the horizontal component

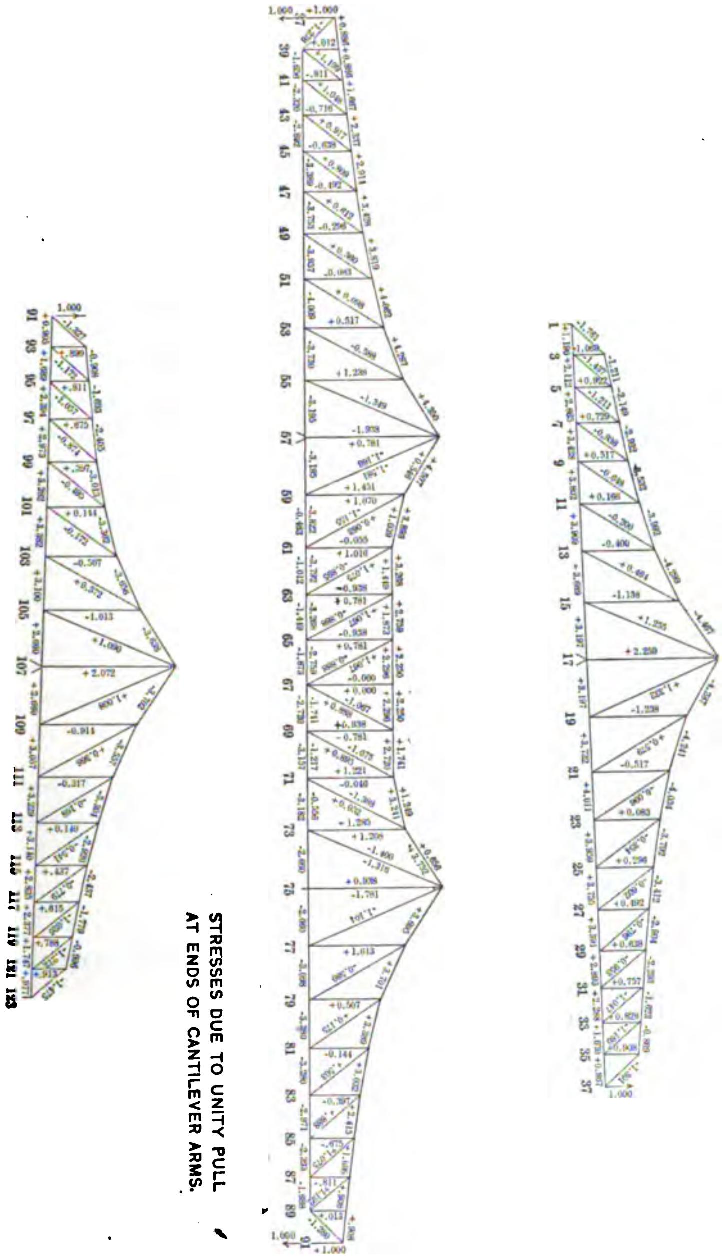
$$H = \frac{\delta}{\delta_{H=1}},$$

necessary to reduce this separation to zero, that is, the real value of H due to this load on the actual structure. Thus we could find the value of H in the actual structure for load unity at each load point, whence, by multiplication by any loads, and summation, the value of H for those loads would follow. And since the actual stress due to any loading will be the stress for that loading, if there were no horizontal component (anchor chain supposed cut), plus the stress due to the actual horizontal component for that loading, or analytically,

$$S = S_0 + H S';$$

the problem of the determination of the stresses in the actual structure for any actual loading would herewith be solved.

In the Blackwell's Island Bridge, suppose we had cut the two "rocker arms" and made their stresses, P_L, and, P_R, the unknown



STRESSES DUE TO UNITY PULL AT ENDS OF CANTILEVER ARMS.

FIG. 8.

forces to be determined. The principles are the same as before, but the work is a trifle less simple, as we have two (unknown) variables instead of one. We have to find the analytic conditions that, under any given load and the corresponding rocker arm stresses, the separation of the cut ends of the rocker arms shall be zero.

First, let us find what displacements of the ends of the cantilever arms (the cut ends of the rocker arms) would result from unity pull at those ends, in the sense of tension in the rocker arms, but on the supposition that both rocker arms are cut. We have, in Fig. 2, the stresses due to such pulls of unity throughout the structure. Let S'_L denote these stresses in the left-hand or Manhattan truss, S'_{LC} those in the center or Blackwell's Island truss for pull of unity on the left (or Manhattan) cantilever end, S'_{RC} those in this truss for pull of unity on the right (or Queens) cantilever end, and S'_R those in the right-hand or Queens truss. The upward deflection of the end of the cantilever of the Manhattan truss, for any elongations, Δ_L, of its bars would be, $\delta_L = \Sigma S'_L \Delta_L$, and, consequently, for the elongations

$$(\Delta_L)_{P_L = 1} = \frac{S'_L l}{E a},$$

due to unity upward pull at the end of the cantilever arm, this deflection is,

$$(\delta_L)_{P_L = 1} = \sum \frac{(S'_L)^2 l}{E a};$$

similarly, the downward deflection of the end of the left cantilever arm of the center span, for any elongations, Δ_C, of the center span bars, is $\delta_{LC} = \Sigma S'_{LC} \Delta_C$; and so, for the elongations,

$$(\Delta_{LC})_{P_L = 1} = \frac{S'_{LC} l}{E a},$$

due to unity downward pull at the end of the left cantilever arm, the downward deflection is,

$$(\delta_{LC})_{P_L = 1} = \sum \frac{(S'_{LC})^2 l}{E a}.$$

In the same way, for the downward deflection of the end of the right cantilever arm of the center span, we have, $\delta_{RC} = \Sigma S'_{RC} \Delta_C$, generally, and

$$(\delta_{RC})_{P_R = 1} = \sum \frac{(S'_{RC})^2 l}{E a},$$

for that due to unity downward pull at the end of the right cantilever arm, and for the upward deflection of the end of the cantilever of the Queens truss, we have, $\delta_R = \Sigma\, S'_R\, \Delta_R$, generally, and for unity upward pull at the end of the cantilever arm,

$$(\delta_R)_{P_R = 1} = \sum \frac{(S'_R)^2\, l}{E\, a}.$$

Finally, the downward deflection of the end of the left cantilever arm of the center span, due to a downward pull of unity on the end of the right cantilever of the center span, is,

$$(\delta_{L\,C})_{P_R = 1} = \sum \frac{S'_{L\,C}\, S'_{R\,C}\, l}{E\, a} = (\delta_{R\,C})_{P_L = 1},$$

which is equal to the deflection of the end of the right cantilever arm of the center span, due to a pull of unity on the end of the left cantilever arm.

Now, let us raise the question, what would be the rocker arm stresses due to a single load, say of unity; applied to any load point of the structure? Let these stresses be P_L and P_R, respectively. Then the actual stresses in the structure may evidently be built up of two parts, S_O, those due to the load alone, considering the rocker arms as cut, and $P_L\ S'_L$ in the left shore span, $P_L\ S'_{L\,C} + P_R\ S'_{R\,C}$ in the center span and $P_R\ S'_R$, in the right shore span. The corresponding elongations of the bars are

$$\frac{S_O\, l}{E\, a},\ P_L \frac{S'_L\, l}{E\, a},\ P_L \frac{S'_{L\,C}\, l}{E\, a} + P_R \frac{S'_{R\,C}\, l}{E\, a},\ \text{and}\ P_R \frac{S'_R\, l}{E\, a}.$$

The consequent separation of the cut ends of the left rocker arm will be

$$O = \delta_L + \delta_{L\,C} = P_L \left[\sum \frac{(S'_L)^2\, l}{E\, a} + \sum \frac{(S'_{L\,C})^2\, l}{E\, a} \right]$$

$$+ P_R \sum \frac{S'_{L\,C}\, S'_{R\,C}\, l}{E\, a} + \left\{ \begin{array}{c} \sum \dfrac{S'_L\, S_O\, l}{E\, a} \\ \text{or} \\ \sum \dfrac{S'_{L\,C}\, S_O\, l}{E\, a}, \\ \text{or} \\ O \end{array} \right.$$

according as the load is on the left, the center or the right truss.

Similarly, the separation of the cut ends of the right rocker arm will be:

$$O = \delta_{RC} + \delta_R = P_L \sum \frac{S'_{LC} S'_{RC} l}{E a}$$

$$+ P_R \left[\sum \frac{(S'_{RC})^2 l}{E a} + \sum \frac{(S'_R)^2 l}{E a} \right] + \begin{cases} O \\ \text{or} \\ \sum \frac{S'_{RC} S_O l}{E a}, \\ \text{or} \\ \sum \frac{S'_R S_O l}{E a} \end{cases}$$

according as the load is on the left, the center or the right truss. In these expressions, the terms for the stretch of the rocker arms must not be forgotten. The solution of these two simultaneous equations which are linear in our only two unknown quantities, P_L, and, P_R, will give their values. By solving these equations, successively, for a load unity at each load point, we could find the values of the rocker arm stresses for each such load, and thence, by summation, for any loads. The actual stresses due to any loading are those which would exist if there were no rocker arms plus those due to the actual rocker arm stresses, or, analytically,

$$\underset{\text{left truss}}{S = S_O + P_L S'_L}, \quad \underset{\text{center truss}}{S = S_O + P_L S'_{LC} + P_R S'_{RC}}, \quad \underset{\text{right truss}}{S = S_O + P_R S'_R},$$

so that the problem for the actual structure and for any loading is thus solved.

We may note that, for any loading, the equations

$$O = \delta_L + \delta_{LC} = P_L \left[\sum \frac{(S'_L)^2 l}{E a} + \sum \frac{(S'_{LC})^2 l}{E a} \right]$$

$$P_R \sum \frac{S'_{LC} S'_{RC} l}{E a} + \sum \frac{S'_L S_O l}{E a} + \sum \frac{S'_{LC} S_O l}{E a},$$

and

$$O = \delta_{RC} + \delta_R = P_L \sum \frac{S'_{LC} S'_{RC} l}{E a}$$

$$+ P_R \left[\sum \frac{(S'_{RC})^2 l}{E a} + \sum \frac{(S'_R)^2 l}{E a} \right] + \sum \frac{S'_{RC} S_O l}{E a} + \sum \frac{S'_R S_O l}{E a},$$

furnish directly, by their solution, the values of P_L and P_R and thence the actual stresses. We may further note the special cases, first, of a load of unity suspended on the end of the left (or Manhattan) truss cantilever arm, for which $S_O = - S'_L$, simply, where we find by solution the rocker arm stresses,

$$(P_L)_L = \frac{\sum \frac{(S'_L)^2\, l}{E\, a}\left[\sum \frac{(S'_R)^2\, l}{E\, a} + \sum \frac{(S'_{R\,C})^2\, l}{E\, a}\right]}{\left[\sum \frac{(S'_L)^2\, l}{E\, a} + \sum \frac{(S'_{L\,C})^2\, l}{E\, a}\right]\left[\sum \frac{(S'_R)^2\, l}{E\, a} + \sum \frac{(S'_{R\,C})^2\, l}{E\, a}\right] - \left[\sum \frac{S'_{L\,C}\, S'_{R\,C}\, l}{E\, a}\right]^2},$$

and

$$(P_R)_L = \frac{-\sum \frac{(S'_L)^2\, l}{E\, a}\sum \frac{S'_{L\,C}\, S'_{R\,C}\, l}{E\, a}}{\left[\sum \frac{(S'_L)^2\, l}{E\, a} + \sum \frac{(S'_{L\,C})^2\, l}{E\, a}\right]\left[\sum \frac{(S'_R)^2\, l}{E\, a} + \sum \frac{(S'_{R\,C})^2\, l}{E\, a}\right] - \left[\sum \frac{S'_{L\,C}\, S'_{R\,C}\, l}{E\, a}\right]^2};$$

and, second, the case of a load of unity suspended on the end of the right (or Queens) truss cantilever arm for which $S_O = -S'_R$, simply, and where,

$$(P_L)_R = \frac{-\sum \frac{(S'_R)^2\, l}{E\, a}\sum \frac{S'_{L\,C}\, S'_{R\,C}\, l}{E\, a}}{\left[\sum \frac{(S'_L)^2\, l}{E\, a} + \sum \frac{(S'_{L\,C})^2\, l}{E\, a}\right]\left[\sum \frac{(S'_R)^2\, l}{E\, a} + \sum \frac{(S'_{R\,C})^2\, l}{E\, a}\right] - \left[\sum \frac{S'_{L\,C}\, S'_{R\,C}\, l}{E\, a}\right]^2},$$

and

$$(P_R)_R = \frac{\sum \frac{(S'_R)^2\, l}{E\, a}\left[\sum \frac{(S'_R)^2\, l}{E\, a} + \sum \frac{(S'_{R\,C})^2\, l}{E\, a}\right]}{\left[\sum \frac{(S'_L)^2\, l}{E\, a} + \sum \frac{(S'_{L\,C})^2\, l}{E\, a}\right]\left[\sum \frac{(S'_R)^2\, l}{E\, a} + \sum \frac{(S'_{R\,C})^2\, l}{E\, a}\right] - \left[\sum \frac{S'_{L\,C}\, S'_{R\,C}\, l}{E\, a}\right]^2};$$

We have now seen how the calculation of an indeterminate structure may be reduced to that of a determinate structure, by doing away with superfluous restraints and cutting superfluous parts, substituting for the influences thus destroyed corresponding replacing forces; how these replacing forces are determined by expressing the condition that the distortions shall remain as before; and how we may most easily express these latter conditions, analytically, by the aid of the principle of virtual velocities, thus obtaining simultaneous linear equations in the unknown replacing forces (equal in number to those forces), whose solution gives the values of these forces and so reduces the calculations to those of determinate structures. And we have followed this process in the case of two particular structures. But much yet remains to be considered in the way of perfecting and completing our procedure.

We have, it is true, already shown how we may obtain an analytic expression for the value of each of the replacing forces for any given loading, and we have given such expressions, or their elements, for the two bridges whose calculation is here used for illustration. But, for such calculation, we wish to be able to determine readily these replacing forces for any loading, and the application of these formulas separately to each loading would be exceedingly laborious, for even a single application involves much labor. Evidently, anything that diminishes this labor will be highly desirable, and, as a first step, we must note the determination of the values of the replacing forces for a load of unity at each of the load points. Then the values for all other loadings follow at once from these by simple multiplication and summation. This procedure would reduce the applications of our formulas to as many loadings as there are load points only, a great saving, but still leaving far too much work to be done. Here, again, another theorem comes to our rescue—that of the reciprocal nature of displacements, often called, in the theory of structures, "Maxwell's reciprocal theorem," and known, in the general theory of the elasticity of solid bodies, as "Betti's reciprocal theorem." Briefly, Maxwell's theorem asserts that the displacement of any point, A, in any given direction, $A\ C$, due to unity force at B in any given direction, $B\ D$, is equal to the displacement of B in the direction, $B\ D$, due to unity force applied at A in the first direction, $A\ C$. The reason for this is exceedingly simple. Let the displacement of A in

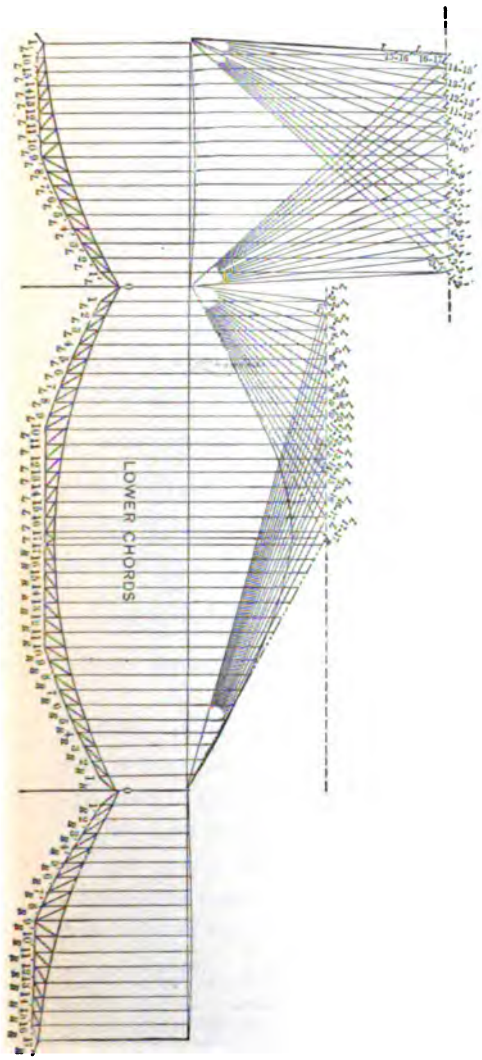


FIG. 4.

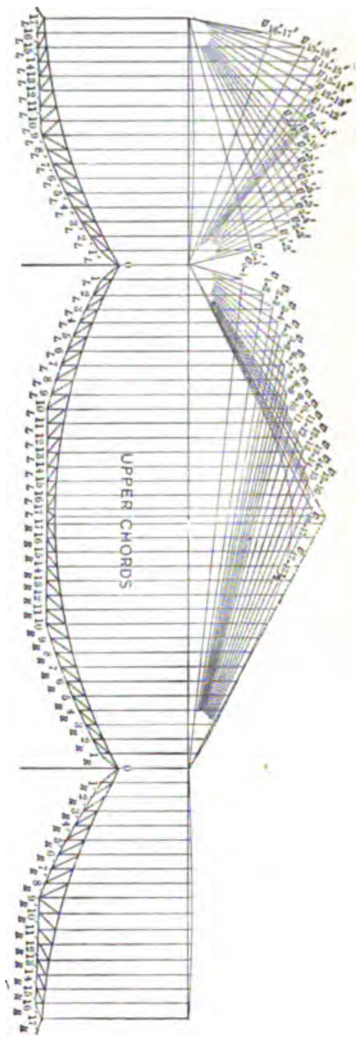


FIG. 3.

the direction, $A\ C$, due to unity force at B be a, and the simultaneous displacement of B in the direction, $B\ D$, of the force be b'. The potential energy of the body will be increased through the application of the force of unity at B by $\frac{1}{2}\ b'$. Now let unity force be applied at A in the direction, $A\ C$, of the displacement, a, and let A suffer the consequent further displacement, a', in the same direction, and B the further displacement, b, in the same direction, $B\ D$, as b'. The potential energy will be further increased by $\frac{1}{2}\ a' + b$, or the total increase from applying both forces of unity will be,

$$\varDelta E_1 = \frac{1}{2}\ a' + \frac{1}{2}\ b' + b.$$

But suppose the forces of unity had each been applied, first at A and then at B. A, under the force there applied, would have been displaced by a', while B was simultaneously displaced by b, the potential energy now increasing $\frac{1}{2}\ a'$. Then, applying the second force of unity at B, it would be displaced by b', while A would simultaneously be displaced by a, resulting in a further increase of the potential energy of $\frac{1}{2}\ b' + a$, and the total increase, from applying both forces in this order, would be $\varDelta E_2 = \frac{1}{2}\ a' + \frac{1}{2}\ b' + a$. But since the order of application of the loads cannot affect the ultimate result, $\varDelta E_1 = \varDelta E_2$, and therefore $a = b$.*

Consider the practical bearing of this theorem on our problem. We wish to know the stress in a certain superfluous member or the restraint acting on a certain part of the structure due to forces applied at any of certain other points which we may term load points. Suppose the member, whose stress is sought, to be cut, or the restraint, whose amount is desired, to be removed (all other superfluous members or restraints remaining unchanged), and a pair of equal and

* The following brief proof based on the principle of virtual velocities is recommended by Prof. George F. Swain, M. Am. Soc. C. E. Let S'_A be the stresses in the bars due to unity force at A in the direction, $A\ C$, and S'_B the stresses in the bars due to unity force at B in the direction, $B\ D$. The displacement of B in the direction. $B\ D$, due to unity force at A in the direction, $A\ C$, will then be, $b = \sum \frac{S'_B\ S'_A\ l}{E\ a}$. But the displacement of A in the direction, $A\ C$, due to unity force at B in the direction, $B\ D$, will be, $a = \sum \frac{S'_A\ S'_B\ l}{E\ a}$, whence $a = b$.

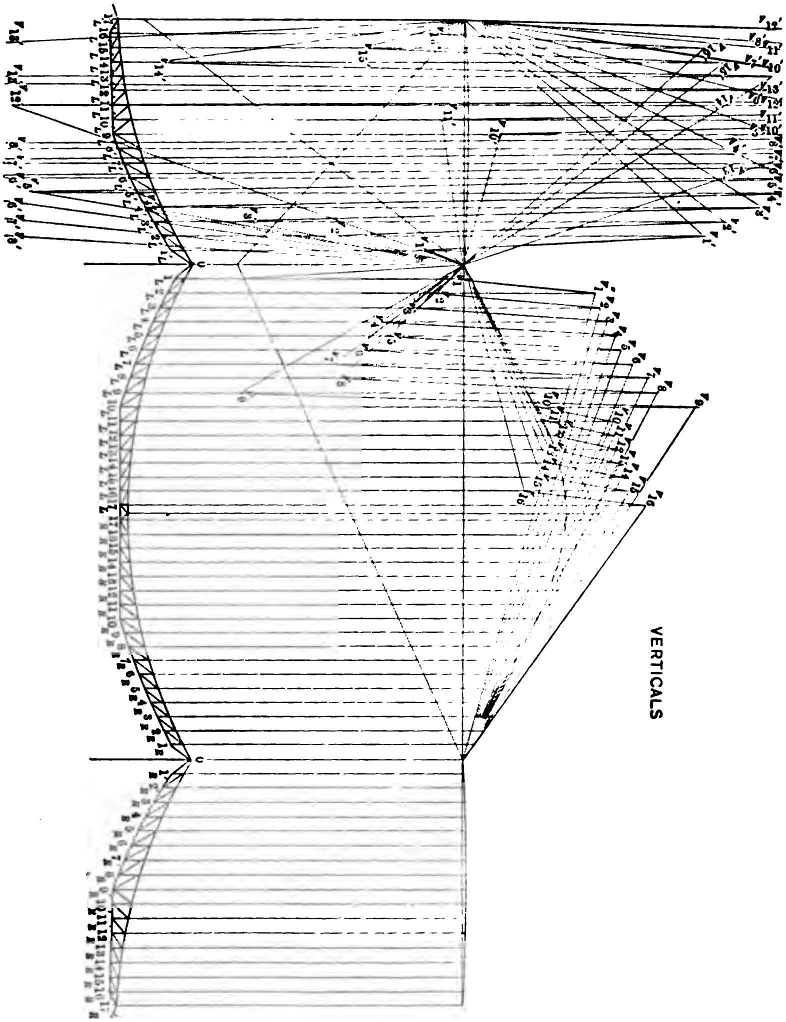


Fig. 3.

opposite forces of unity to be applied at the cut ends or in place of the removed restraint. Through the methods already described, we can determine the consequent stresses, and therefore strains, of all members due to the pair of forces of unity. Suppose we now determine the consequent displacements of all the load points and also the separation of the ends of the cut member, or of the points between which the removed restraint acted. Evidently, by Maxwell's theorem, the separation of the ends of the cut member (or of the points between which the removed restraint acted) due to unity force applied at any load point, would be equal to the displacement of that load point in the direction of this applied force, due to the pair of forces of unity. And this quantity, divided by the separation of the cut ends (or of the points between which the removed restraint acted), due to the pair of forces of unity, would give the value of the stress or restraining force sought.

The problem is now reduced to that of obtaining a displacement diagram of the load points for a known set of strains. There are many methods by which this may be done, among others that of the successive application of the principle of virtual velocities to each load point; but even this is unnecessarily long and laborious, unless exceptional accuracy is desired. Far simpler and briefer, in the case of frameworks, are certain graphical methods which have now to be described.

The first of these is the Williot displacement diagram. Suppose that from any two points, A and B (Fig. 7), two bars, $A\ C$ and $B\ C$, extend, meeting in C. Suppose that A and B each suffer certain displacements, and, further, that $A\ C$ and $B\ C$ suffer certain changes in length, and that we have to find the resulting displacement of the joint, C.

Imagine $A\ C$ and $B\ C$ freed at C. Let $A\ A'$ be the given displacement of A, and $B\ B'$ that of B. Conceive $A\ C$ moved parallel to itself to $A'\ C'_A$, and, similarly, $B\ C$ moved to $B'\ C'_B$. Let $C'_A\ C''_A$ be the change in length of $A\ C$, and $C'_B C''_B$ that of $B\ C$. Evidently if we now swing $A'\ C''_A$ about A' and $B'\ C''_B$ about B' until the free ends meet at C''', this will be the new position of the joint and $C\ C'''$ will be the displacement sought. Practically the displacements, $A\ A'$ and $B\ B'$, and the changes in length, $C'_A\ C''_A$ and $C'_B\ C''_B$, will be extremely small compared with the lengths, $A\ C$ and $B\ C$, and the arcs

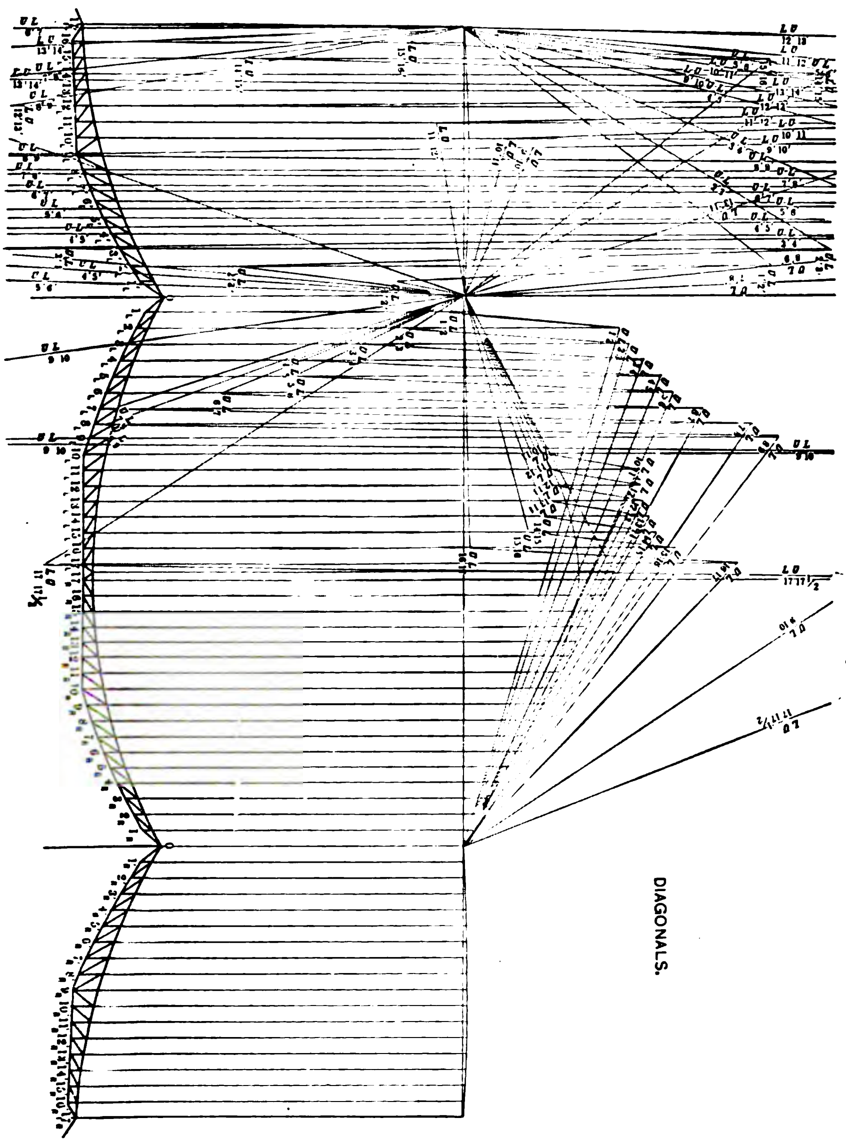


FIG. C.

DIAGONALS.

of circles, $C''_A\ C'''$ and $C''_B\ C'''$, will practically coincide with the perpendiculars from C''_A and C''_B to the directions of $A\ C$ and $B\ C$, respectively. Moreover, we note that the only essential part of the construction is the figure $C\ C'_A\ C''_A\ C'''\ C''_B\ C'_B$, which is shown here alone in Fig. 8. This, then, is all we need draw. And, since exaggeration does not affect its proportions, we may make it of any convenient size. Thus, from a point, C, we lay out the displacements of A and B at a thousand times their real amount, or $C\ C'_A$ and $C\ C'_B$. From C'_A and C'_B we lay off, in the directions of the bars (which remain practically unchanged) their changes in length, $C'_A\ C''_A$ and $C'_B\ C''_B$, respectively, also exaggerated a thousand times. From C''_A and C''_B we draw lines, $C''_A\ C'''$ and $C''_B\ C'''$, perpendicular in direction to the directions of the bars, and their intersection, C''', furnishes us the displacement, $C\ C'''$, of C in amount (exaggerated a thousand times) and in direction. The method is exceedingly simple and, when the intersections are good, is exact. When the intersections are not good, the construction itself is likely to be bad.

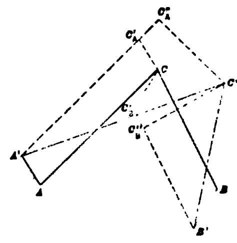


FIG. 7.

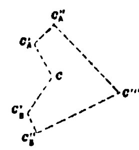


FIG. 8.

This method, applied repeatedly, enables us to find the displacement of all joints of any framework which can be built up by adding successive pairs of bars, for any given set of strains. We must start by assuming a fixed point and a fixed direction and build upon these. The various displacements, thus found, will then have to be corrected, generally, for the error in the assumption that a certain direction was fixed. This is most readily done by noting the true displacement of some point, which is usually obvious, and proportioning the corrections of all other points to its correction, a simple matter, since it involves simply the corrections due to a small rotation of the framework. The displacement of each joint, due to the rotation of the framework about any point, is, in direction, perpendicular to the line connecting the point with that joint and, in amount, proportional to the length of this connecting line. If, therefore, we draw a figure similar to our framework, but rotated 90°, in the sense of the correct-

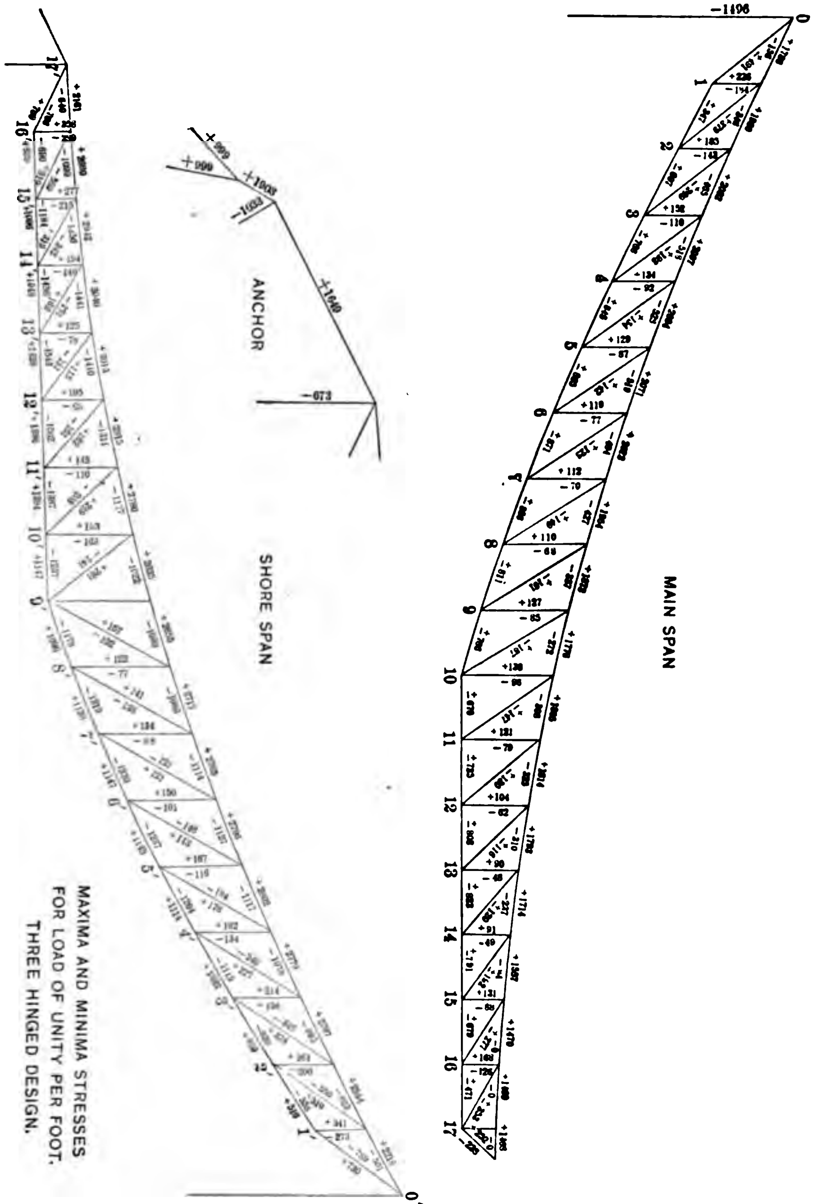
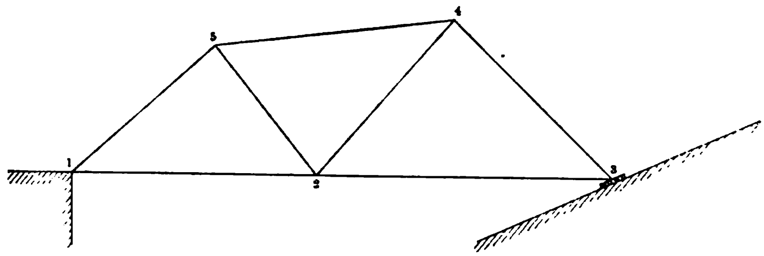

MAIN SPAN
SHORE SPAN
ANCHOR
MAXIMA AND MINIMA STRESSES
FOR LOAD OF UNITY PER FOOT.
THREE HINGED DESIGN.

Fig. 9.

ing rotation, with one of its points, which is actually fixed, coinciding with the displacement position of that point in the Williot diagram, and to such scale as shall make the line from any other of its points to the corresponding displacement position of that point in the Williot diagram correctly represent the actual direction of the displacement of that point, then the points of this figure, thus drawn, will give us, by their distances and directions from the actually fixed point, the



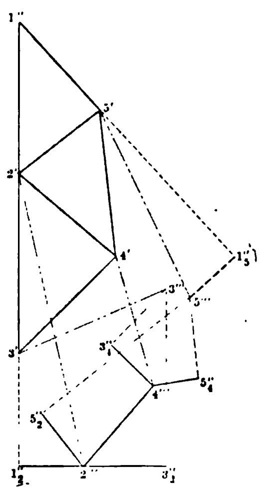


FIG. 10.

rotation displacements of all these points. The distance and direction from any one of the joints of this figure to the corresponding displacement position of the point in the Williot diagram will give the actual displacement in amount and direction (all, of course, to the exaggerated scale).

For example, consider the simple truss in Fig. 10 supported at 1 and on rollers at 3. Suppose we regard 2 as a fixed point and 2–4 as a fixed direction, and, starting from $2'''$, proceed to the corresponding Williot displacement diagram. Suppose 2–4 to have the extension $2'''$–$4'''$. The extension, $2'''$–$5''_2$, of 2–5 and compression, $4'''$–$5''_4$, of 4–5 enable us to find the corresponding displacement position, $5'''$, of 5, whence the extension, $2'''$–$1''_2$, of 2–1 and the compression, $5'''$–$1''_5$, of 5–1 enable us to find the displacement position, $1'''$, of 1. Lastly, the extension, $2'''$–$3''_2$, of 2–3 and the compression, $4'''$–$3''_4$, of 4–3 give us the displacement position, $3'''$, of 3. That is to say, $2'''$–$1'''$ represents the magnitude and direction of the motion of 1, $2'''$–$5'''$ that of 5, $2'''$–$4'''$ that of 4, and $2'''$–$3'''$ that of 3, all with

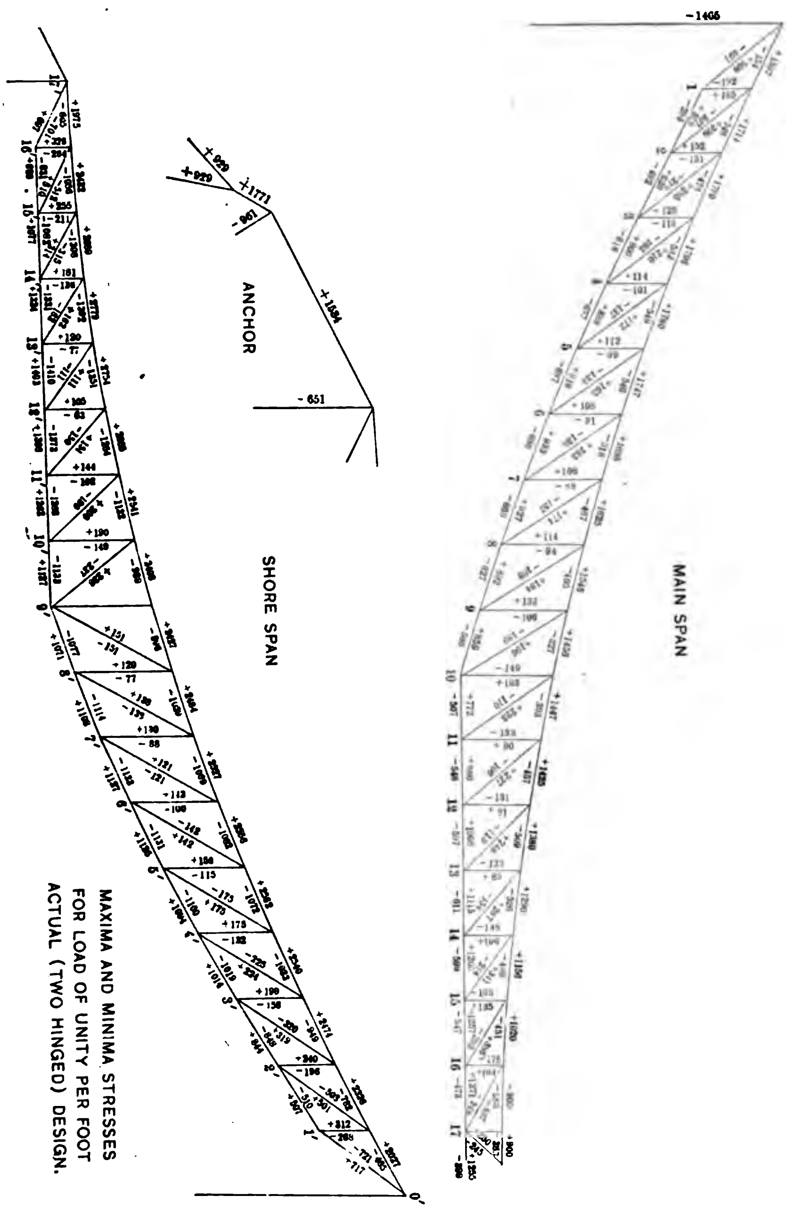

ANCHOR
SHORE SPAN
MAIN SPAN
MAXIMA AND MINIMA STRESSES
FOR LOAD OF UNITY PER FOOT
ACTUAL (TWO HINGED) DESIGN.

Fig. 11.

reference to 2 as a fixed point and 2–4 as fixed in direction. Now the actual displacement of 1 is zero, and the actual displacement of 3 is in the direction of the plane of its roller bed, instead of the direction, 2‴–3‴, just found. Thus, the actual displacement of 3 must be found by combining its displacement, 1‴–3‴, with respect to 1 on the supposition that 2–4 is a fixed direction, with such a rotational displacement about 1 as shall give the actual displacement in the roller bed direction. This rotational displacement of 3 occurs perpendicular to the line 1–3, therefore we find the intersection, 3′, of the line, 1‴–3′, perpendicular to 1–3 with the line, 3‴–3′, in the roller bed direction, and 3′–3‴ is the actual displacement of 3 in amount and direction. Now we construct the figure 1‴ 2′ 3′ 4′ 5′, similar to the original truss figure, 1 2 3 4 5. Then, 2′–2‴, 3′–3‴, 4′–4‴ and 5′–5‴, will be the actual displacements of the points, 2, 3, 4, and 5, in amount and direction (to the exaggerated scale).

The Williot displacement diagram thus furnishes an exceedingly simple and beautiful method of attaining the desired end. But, practically, this method has defects. It tends to sum up and multiply errors, even more than does the use of the ordinary Cremona stress diagram. And if the elongations of the members are laid out on a reasonable scale the displacement diagram as a whole becomes excessively large. By making several independent diagrams, especially with different points and lines assumed as fixed, and averaging the results, the first of these objections can be overcome in part, but for greater accuracy another procedure is often preferable.

In bridges, the load points are, as a rule, either the top or bottom chord joints or are connected, simply, with them. All that is needed in such a case, then, is the "deflection curve" of the "load line" for unity value of the replacing force under consideration. This curve permits of comparatively accurate and ready determination in a variety of ways, but, in the case of frameworks, probably the best way is, first, to determine the deflection angle at each "panel point" of the load line, from which the deflection curve readily follows. To determine these angles (say it is the angle between two adjacent chord members, in each case), we have to determine the distortion angles of the triangles

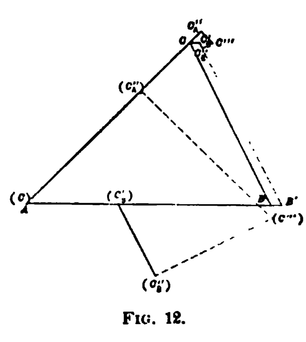


FIG. 12.

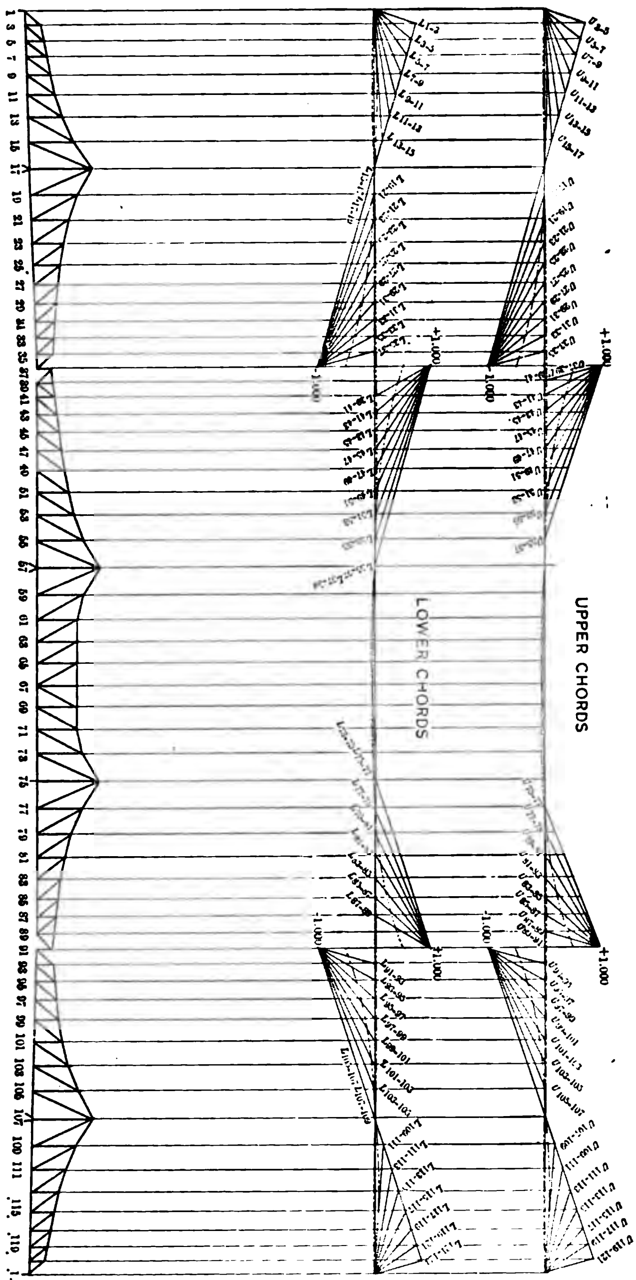


Fig. 13.

whose sides are in part the members meeting at the joint. And these distortion angles, we may very readily determine by the aid of Williot polygons, as follows: Let $A\ B\ C$ (Fig. 12) be such a triangle and the distortion angle at B for given elongations of $A\ B$, $B\ C$ and $A\ C$ be required. Let $A\ B$ increase by $B\ B'$; let $A\ C$ increase by $C\ C''_A$, and let $B\ C$ shorten by $C'_B\ C''_B$. Then C''' will be the resulting position of C. But this means that $B\ C$ will have swung through the small angle $\frac{C''_B\ C'''}{B\ C}$. We have then only to determine, accurately, $C''_B\ C'''$, in order to solve the problem. Practically, we lay out $C\ C''_A$ and $B\ B' = C\ C'_B$ from A on the triangle, $A\ B\ C$, to an exaggerated scale, as $(C)\ (C''_A)$ and $(C)\ (C'_B)$. Then we draw $(C'_B)\ (C''_B)$ parallel to $B\ C$ and equal to $C'_B C''_B$ on the same scale. The perpendicular to $A\ C$ through (C''_A) and that to $B\ C$ through (C''_B) give (C''') and thus $(C''_B)\ (C''')$ which, measured on the exaggerated scale, is the quantity sought. The triangle, $A\ B\ C$, on which this work is done, would preferably be a part of the figure of the truss, carefully and accurately drawn. Proceeding thus, from triangle to triangle, we readily obtain these distortion angles and thus, by summation at each joint, carefully observing signs, the deflection angles at each joint. It now only remains to find the corresponding deflection curve.

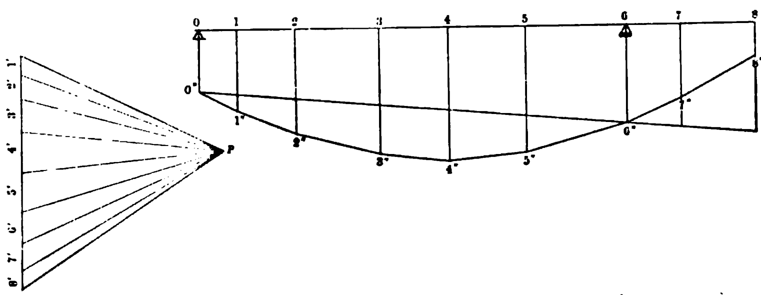


FIG. 14.

The deflection angles thus being known, the deflection curve may be easily found, either graphically or analytically. The former method is a trifle simpler and, generally, is amply accurate. We proceed as follows: Suppose we have a straight horizontal load line, as is practically the case in most bridges. On a vertical we carefully lay

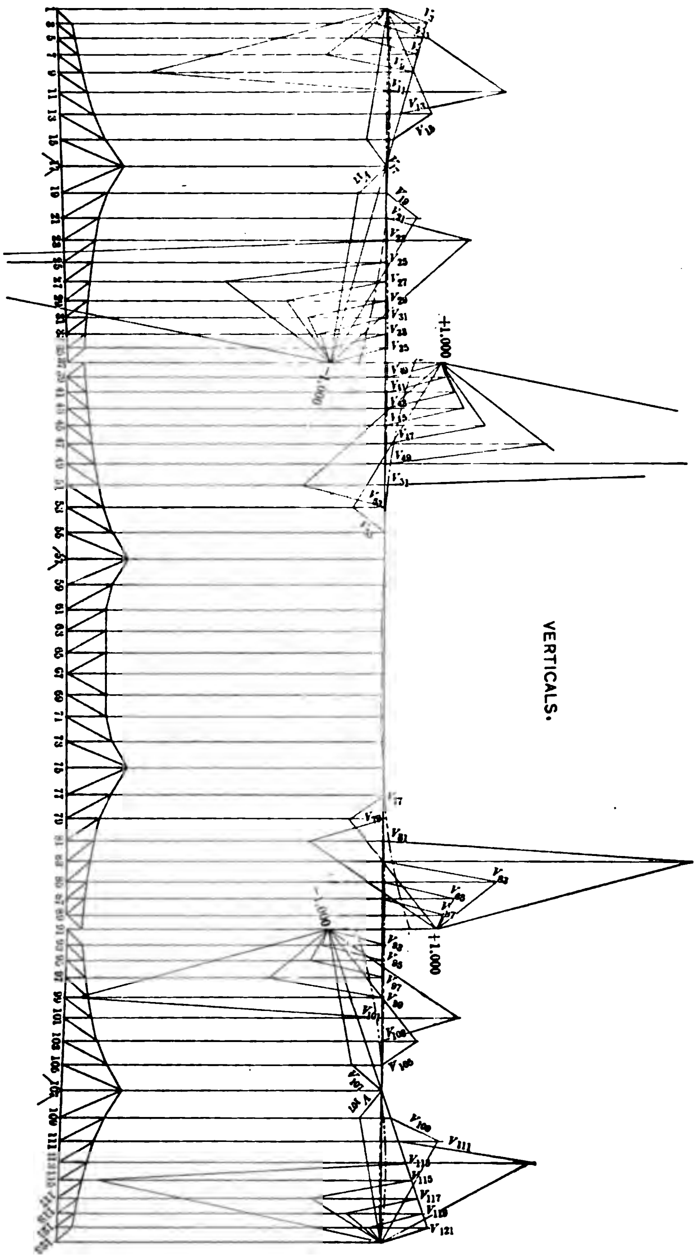


FIG. 15.

off successive distances proportional to the successive deflection angles, preferably by adding the figure values of the angles to one side and plotting the sums all from one point, to avoid the accumulation of errors. Now, if lines were drawn from these points to a point at a suitable (great) distance, the angles between them would be the deflection angles. Instead, we take a comparatively near point or "pole," by which the tangents of the angles are multiplied in a corresponding degree, and draw radiating lines to the set of points. Now, parallel to these lines from the pole, we draw, from panel vertical to panel vertical of the load line, lines of the deflection curve, which is thus at once obtained on the exaggerated scale. We have only to note on this curve the points of no deflection, draw straight lines through them, and from these measure the deflections. They will be the desired deflections to the exaggerated scale.

Thus, let 1, 2, 3, 4, 5, 6, 7, 8 (Fig. 14) be the panel points of the load line, and let 1′, 2′, 3′, 4′, 5′, 6′, 7′, 8′ be proportional to the deflection angles at these points. Let P, at a convenient distance, be taken as a pole and by its aid the curve (polygon), 1″ 2″ 3″ 4″ 5″ 6″ 7″ 8″, be drawn with its sides ("strings") parallel to the corresponding "rays" from P. This curve will be, to an exaggerated scale, the deflection curve sought. Let the points, 0 and 6, of the load line be points of no deflection; then the actual deflections must be measured from the line, 0″ 6″, to the points of the deflection curve. Questions of scale must be carefully looked into, of course, but this is a matter of detail for which we are not warranted in taking space here.

This completes the description of the general method to be pursued in the complete determination of the replacing forces for all load points. Let us briefly note what this work was in the cases of the Manhattan and the Blackwell's Island Bridges.

In the case of the Manhattan Bridge we had the value of but one replacing force to determine for each load point, namely, the value of the "horizontal component." Cutting the anchor chain and applying a pair of tensions, of unity horizontal component, to the cut ends, a set of stresses (and therefore strains) resulted, which we have already noted, and, consequently, a certain deflection curve of the load line. This was found by determining, first, the distortion angles of each of the triangles which formed the trusses, resulting from the strains due to unity horizontal component, a Williot polygon being drawn for

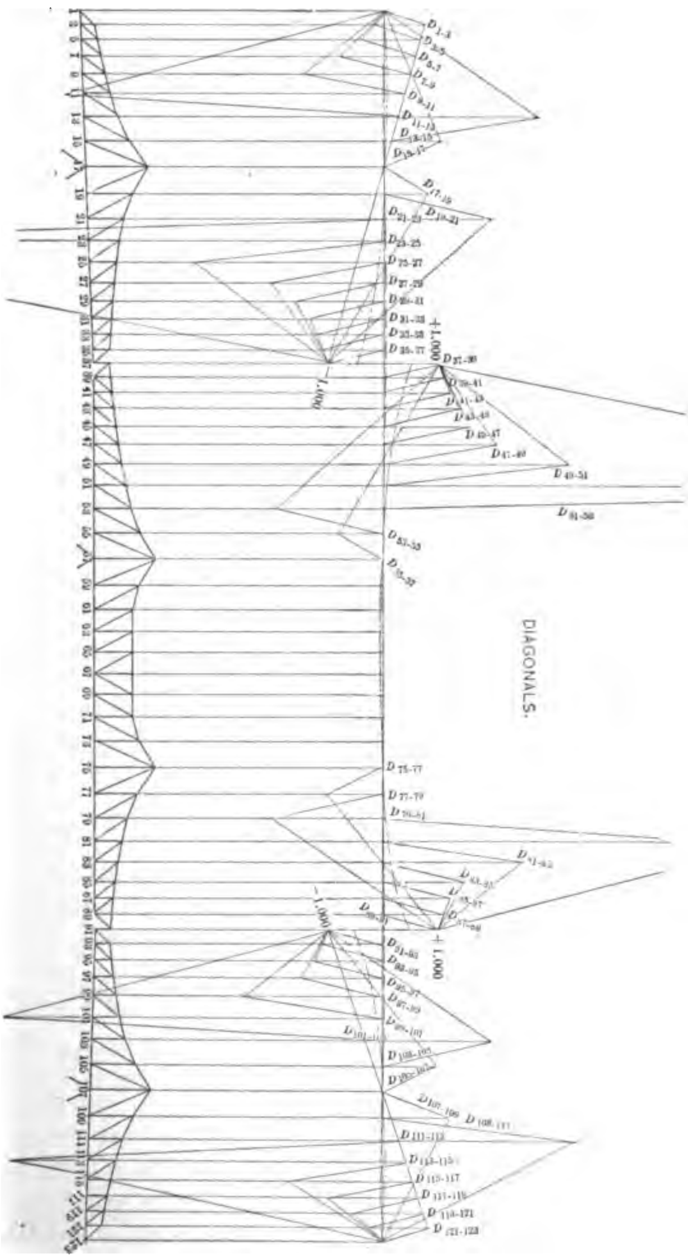


FIG. 16.

each triangle. Thence the deflection angles of the bottom chords were found by simple summation of the distortion angles at each joint. Thence the deflection curve of the bottom chord was found, which would be the same, in this case, as that of the load line. But, as the bottom chord was, in some parts, neither practically straight, nor horizontal, it was necessary that certain small corrections (equal in each panel to the vertical component of the shortening of the chord bar in that panel) be added to the deflections which were found solely from the deflection angles as hitherto described. The final result was a load-line deflection curve for each of the side spans and the center span, whose ordinates were strictly proportional to the value of the horizontal component in the bridge due to load of unity at those points. Dividing these ordinates by the separation of the cut ends of the anchor chain, which unity horizontal component would overcome, previously analytically determined, the actual values of H for load unity at the various panel points were obtained. These values, plotted, gave what is known as the "H curve" or "influence line" for H, which is shown in the illustrations for the influence lines.

In the case of the Blackwell's Island Bridge the procedure was quite similar, but a trifle less simple, for there were two indeterminate forces to consider. The deflection angles and curves were determined, as before, for each main panel point, and for unity pull at the end of each cantilever arm. Now, the left rocker arm alone being supposed cut, and a pair of equal and opposite pulls of unity applied to the cut ends, a certain consequent stress, $\frac{(P_R)_L}{(P_L)_L}$, resulted in the right rocker arm, and the actual deflections were as follows: For the Manhattan truss, those due to unity upward pull at the left rocker arm; for the Blackwell's Island truss those due to unity downward pull on the left rocker arm plus those due to $\frac{(P_L)_R}{(P_L)_L}$ on the right rocker arm; and, finally, for the Queens truss, those due to $\frac{(P_R)_L}{(P_L)_L}$ upward pull on the right rocker arm. The influence line ordinates of the left rocker arm stress were proportional to these ordinates, and since the actual values for load unity at the left and the right rocker arm points were $(P_L)_L$ and $(P_R)_L$, all other values thence followed at once. The procedure for the influence line for the right rocker arm stress was entirely similar, but was here based on unity pull on the right rocker arm and $\frac{(P_L)_R}{(P_R)_R}$ on the left rocker arm.

This, in brief, covers the work necessary for the full determination of the replacing forces or the "indeterminate elements," as they are somewhat contradictorily called. From this point any usual methods of calculation become applicable, but the writer desires to insist on the advantages of continuing the work of actual calculation by the use of influence lines, as was done in the two cases here described. There are also certain important, although simple, modifications of the influence line method, here used, which are worth a little further description and the serious consideration of all who may ever undertake such work.

It may be assumed, of course, that all are familiar with the nature of influence lines and their use, but, for completeness, we may here describe an influence line as one, plotted from a given base line, which corresponds to the load line of the structure, with ordinates at each point of this base line equal (or proportional) in value to the influence of a load of unity at the corresponding point of the load line of the structure, on the part or member to which the influence line refers. The "influence" in question is ordinarily the stress at the part or in the member concerned, but it might as well be a strain or a displacement.

It will be assumed here that the reader is familiar with the methods of drawing and checking normal influence lines of statically determined structures, which are ordinarily composed of not more than three or four straight lines. We will therefore consider only influence lines for statically indeterminate structures.

The normal influence line of a statically indeterminate structure, often referred to as a curve because it is made up, at most, of straight lines, from panel point to panel point only, may be drawn by constructing first the simple influence line of the part concerned for the statically determined structure formed when the superfluous restraints are supposed removed and the superfluous bars cut. To each ordinate of this line may then be added (algebraically) the influences of the replacing forces on the part concerned, for load unity at the point of the structure corresponding to that of the influence line for which the ordinate is being determined. For this purpose we should first prepare an influence line of each of the replacing forces, as previously explained, and a diagram or table for each of the replacing forces, giving the "influence" of the replacing force, when of unity value, on each part of the reduced statically determined structure. These diagrams or tables would give the values of the stresses, S', previously mentioned, and so would already be at hand. The figuring of the "replacing forces corrections" of the statically determined influence line of a given part, then, simply involves multiplying the ordinate of the influence line of each replacing force at the load point for which we are determining the ordinate, by the value of the "influence" of that replacing force on the part whose influence line we are constructing, as shown by the diagram or table, and summing, algebraically, the corrections thus found for all the replacing forces. Nothing further is essential in this connection; but we may note an important variation on the ordinary influence line, which it is often or usually desirable to introduce. Instead of making up the ordinates of the final influence line by adding (algebraically) these replacing force corrections to the ordinates of the "statically determined influence line," we may lay off these corrections from the base line in the opposite sense, so that the ordinates between the line thus obtained and the statically determined influence line will have the final or total values. The correction line thus becomes, practically, the zero or base line.

When we have several replacing forces all influencing a given part, this is about as far as we can go in simplifying the work of constructing the influence lines. The use of these lines (when once constructed) gives at once the kind and amount of the influence of a load of unity at any point of the load line, and so permits us to determine at once where and how to place the loads for influence of a given sign (in particular, what loading to use for maxima and minima), and, then, to determine, by simple multiplication of each load into its corresponding influence line ordinate, and, by algebraic summation of the products, the consequent total influence. But in many, if not most cases, we have only one replacing force and, therefore, only one replacing force correction concerned with the part in question. Here a more brief and simple procedure becomes applicable, which may be called the method of distorted influence lines. If, under these circumstances, we proceeded in the ordinary manner, but plotted the

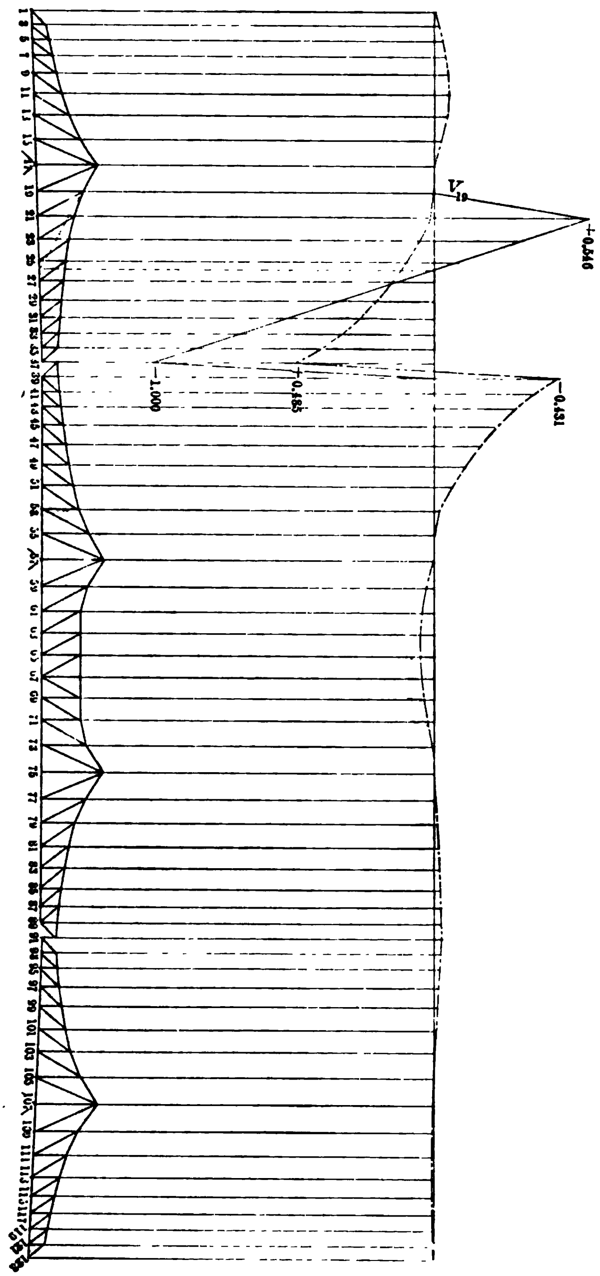


FIG. 17.

replacing force correction line as a zero or base line, as already described, this correction line would differ from the influence line of the replacing force only in having its ordinates different in a fixed proportion, that of the "influence" of unity value of the replacing force, on the part in question, or the influence coefficient. Instead, then, of calculating the ordinates of and plotting the replacing force correction line, we may more easily reduce the statically determined influence line by dividing its ordinates by the influence coefficient and plotting it in connection with the replacing force influence line. Then it combines with that influence line exactly as the normal statically determined influence line and the replacing force correction line would combine to give the true influence ordinates, except that the ordinates here are all reduced by division by the influence coefficient. But points of zero influence are given precisely as before, and the use of the distorted influence line is precisely the same and yields precisely the same results as does the use of the normal line, except that all ordinates and, therefore, all areas must be multiplied by the influence coefficient to give the true results.

But what is the advantage of this procedure? First, if there were but the single part we are considering, to which to apply the method (since we already have the influence line of the replacing force, and the statically determined influence line of our part is comparatively very simple), it is much easier to reduce the statically determined influence line of our part to, and combine it with, the replacing force influence line than to follow the reverse procedure. But, generally, the part is but one of many, all requiring similar treatment, and, since all of them can have their distorted statically determined influence lines plotted in combination with the one replacing force influence line, the advantage rapidly increases in proportion to the number of parts calling for treatment. And the saving of time and labor shows, not alone in plotting the influence lines, but even more in their subsequent use, in the determination of areas. The saving in the cases of the Manhattan and the Blackwell's Island Bridges from the use of distorted, instead of normal, influence lines was enormous. And the complication thus introduced was insignificant.

Consider our two illustrations. In the case of the Manhattan Bridge the replacing force was the stress in the cut anchor chain, or,

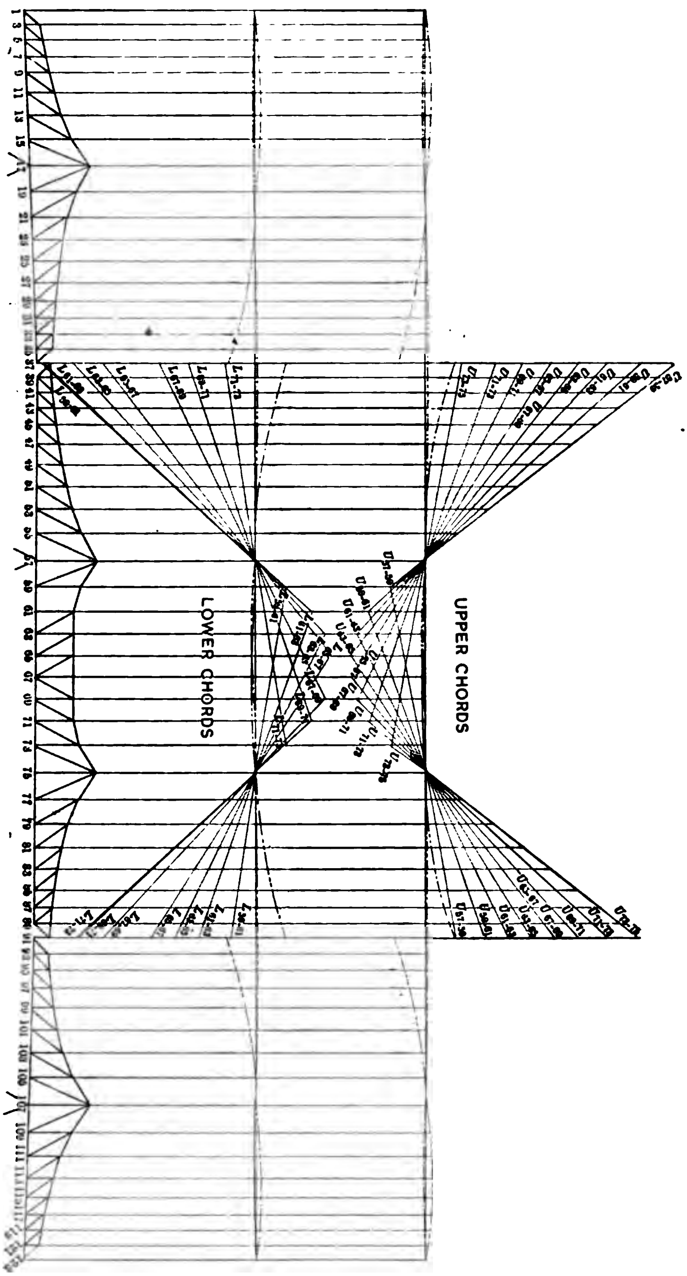


FIG. 18.

rather, its horizontal component. The influence line for this (or the H curve as it is called) was determined once for all and plotted but four times—on the sheet of top chord influence lines, on that of bottom chords, on that of verticals, and, finally, on that of diagonals. Then the distorted statically determined influence lines were drawn. Those for the bottom chords were exceedingly simple and regular. Those for the other members presented no real difficulties. The appearance of the four resulting sheets of influence lines can be seen from Figs. 3, 4, 5 and 6. One other feature of this case may be noted. The H line for a three-hinged design, with center hinge at the top chord, was also drawn, making all influence lines, with respect to it, influence lines for a three-hinged design. All the distorted statically determined influence lines were tested, first by the usual tests, on the supposition of three simple spans. They were then tested in connection with the H line for the three-hinged design. Finally, all calculations were made first for the three-hinged design, and the final corrections were those for the center bottom chord stress as a replacing force, instead of the anchor-chain horizontal component. This made the indeterminate corrections much smaller, and therefore easier to determine for the same degree of accuracy. It also furnished a set of maxima and minima stresses for the three-hinged design, as well as for the two-hinged design, the comparison of which may be made from the stress sheets for loading of 1 000 lb. per ft. given in Figs. 9 and 11.

In the case of the Blackwell's Island Bridge the replacing forces were the stresses in the so-called "rocker arms." Except for members of the center span of the Blackwell's Island truss, the stresses in the members were made up of a statically determined component and but one rocker arm correction. The entire Manhattan truss and the Manhattan cantilever of the Blackwell's Island truss involved only the left rocker arm stress correction. The influence line for the left rocker arm (indicated by a dash and one dot) having been found, all statically determined influence lines of these portions were distorted to combine with this rocker arm influence line, and the results are seen (some details excepted) in the left-hand portions of the distorted influence line sheets (Figs. 13, 15 and 16). The right-hand portions show the corresponding distorted influence lines for the Queens truss and the Queens cantilever of the Blackwell's Island truss.

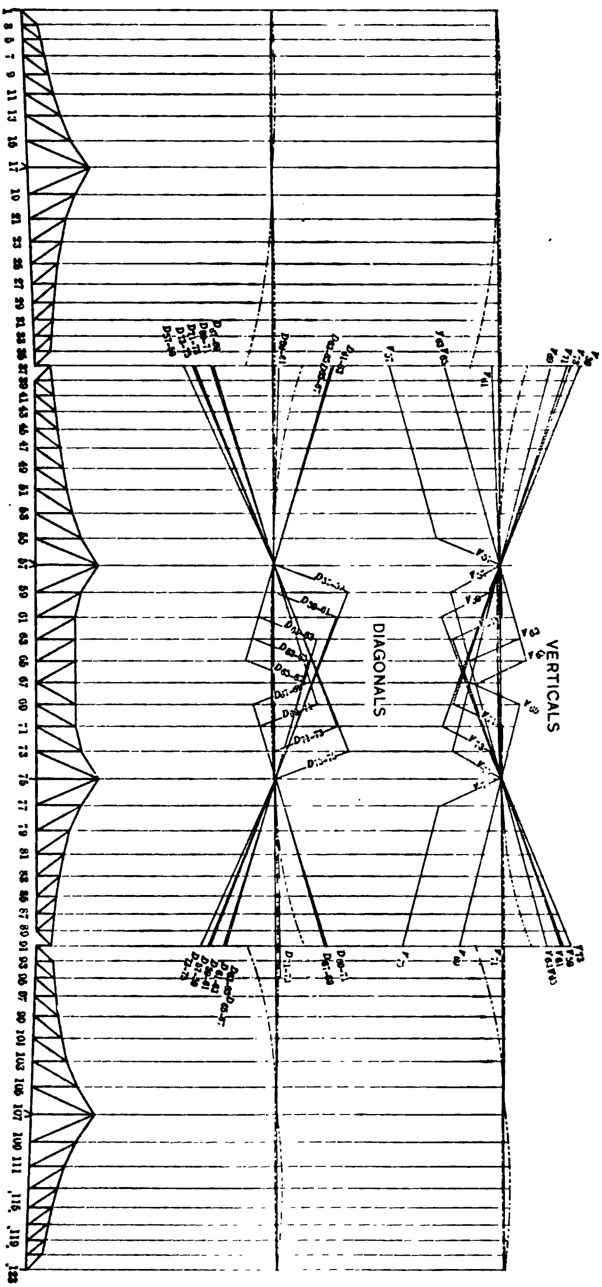


FIG. 19.

based upon the right rocker arm stress (whose influence line is indicated by a dash and two dots). The distorted influence line of V_{19} is enlarged and shown separately in Fig. 17. The center span of the Blackwell's Island truss, involving a correction to the statically determined component from each rocker arm, requires a special rocker arm correction curve to be calculated for each member, to give the true influence for each load point. But this was not actually necessary in the calculations. Normal influence lines were used here (see Figs. 18 and 19).

This completes, in brief form, the description of the essential features of the methods used in making the check stress calculations. Much has been omitted which would have been pertinent and interesting were this a description of the actual work, instead of the methods, simply,—as, for instance, the treatment of certain special cases, many labor-saving devices, the nature of the checks introduced to guard against error and verify the conclusions, and, finally, the numerical values themselves and all that might result from their discussion. But all this would have exceeded the original scope of this paper and only distracted attention from its ends. Beyond mentioning that all the results were checked by the writer by independent methods and calculations which gave exceedingly close agreements, the writer considers it desirable to reserve all such matter for some future occasion.

Vol. XXX. AUGUST, 1904. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

A RATIONAL FORM OF STIFFENED SUSPENSION BRIDGE.

By Gustav Lindenthal, M. Am. Soc. C. E.

To be Presented September 21st, 1904.

That a suspension bridge can be made as rigid as any form of metal bridge, be it arch, cantilever, or truss, is now no longer disputed, and the old prejudice against its supposed unfitness for concentrated loads may be said to have died out.

All the older suspension bridges are limber and undulating structures, without adequate provision against deformation under moving loads. The belief became general that they could not be made as rigid as other bridge systems, and hence they acquired a bad reputation for railroad purposes.

Therefore, when the writer, more than sixteen years ago, proposed a stiffened suspension bridge, of 3 100 ft. span, over the North River, for fast railroad trains, the proposition was received with much doubt and criticism. This, however, had the good effect of inducing discussion and investigation, and of gradually clearing away much of the fog which surrounded the theory of rigidity.

Rankine was the first mathematician (in 1869) to publish a rational theory for the adequate dimensioning of stiffening trusses. His formu-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

las were at first believed to give too heavy trusses, as compared with the makeshifts in vogue, and for that reason they were rarely used. For instance, they were not used in the plans for the Brooklyn Bridge.

Theoretical refinement has led to somewhat lighter stiffening trusses than were required originally by his theory, which had ignored that portion of the live load absorbed by the deformation of the cable. The saving in weight is not great enough, however, to justify the belief that a properly designed suspension bridge of that kind represents a striking economy over the cantilever, arch and truss, except for very long spans (say of 2 000 ft. and more), when the dead load of the structure is several times greater than the live load.

The term "properly designed" is used advisedly, for in no other bridge system is it possible to use such attenuated dimensions, to have so much deflection up and down, and to commit such errors of design, as in the suspension system, without inviting instant collapse of the structure.

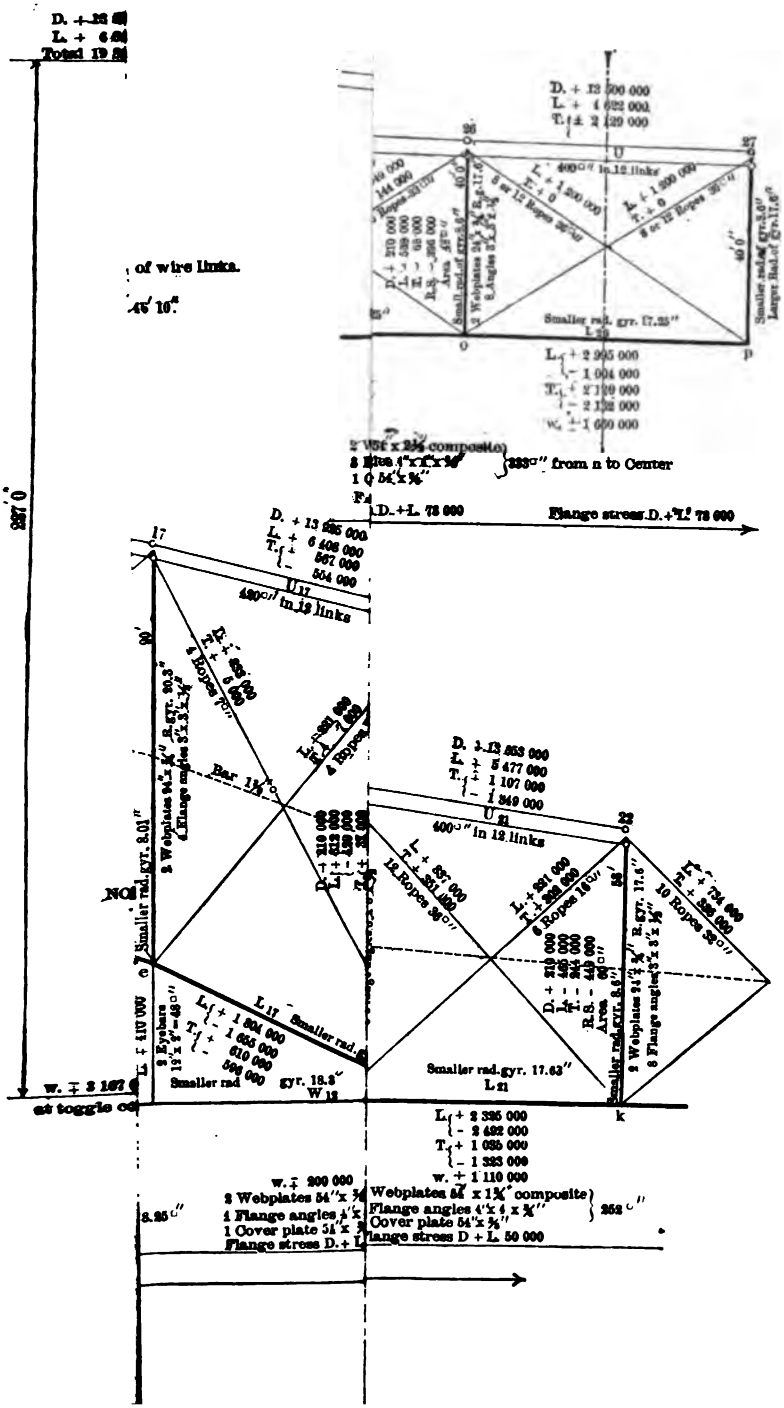
As an illustration in point, compare the cantilever bridge over the Forth in Scotland, having two main spans, each of 1 700 ft, with the Brooklyn Suspension Bridge, having one main span of 1 600 ft. If one of the main spans of the Forth Bridge were loaded on both tracks with locomotives and the other spans were not loaded, the structure would still be perfectly safe. If the main span of the Brooklyn Suspension Bridge were loaded on its four tracks with the full cars in daily use, close together, and the side spans were not loaded, then the cables, otherwise strong enough, would sag so much that they would wreck the stiffening trusses, if not the entire structure.

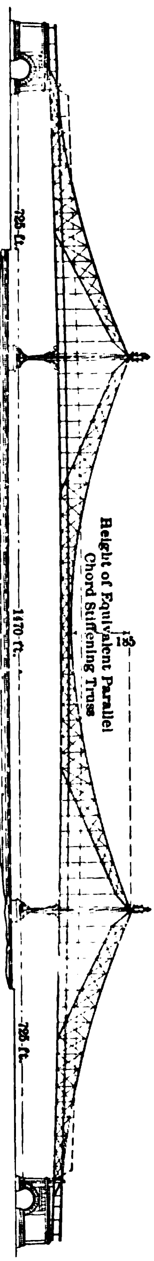
Another prejudice, originating with weak and inadequately stiffened suspension bridges, is the rule prohibiting marching in step, on the ground that the rhythmical cumulative vibrations might eventually break down the structure, whether weak or strong. The fabled fiddler would have to be longer lived than iron and steel efficiently protected against corrosion, before he would succeed with a modern railroad bridge, or equally well designed highway bridge.

A suspension structure, to be comparable with other bridge systems, therefore, must be dimensioned on the same conditions of strength and stability. When that is done, its vaunted economy vanishes, except for very long spans, as before mentioned.

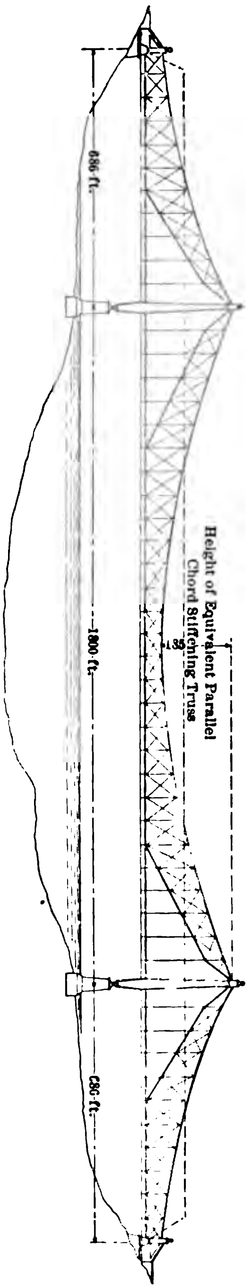
The only recommendation of stiffening trusses is that of a convenient construction independent of the cable, which can be made

PLATE XXXI.
PAPERS, AM. SOC. C. E.
AUGUST, 1904.
LINDENTHAL ON
STIFFENED SUSPENSION BRIDGE.





MANHATTAN BRIDGE OVER THE EAST RIVER IN NEW YORK.



STIFFENED RAILROAD SUSPENSION BRIDGE
OVER THE ST. LAWRENCE RIVER AT QUEBEC, PROPOSED IN 1898.

FIG. 1.

more or less efficient or costly as may suit the fancy of the designer, or the financial resources of the owners, frequently on the principle that "half a loaf is better than no bread."

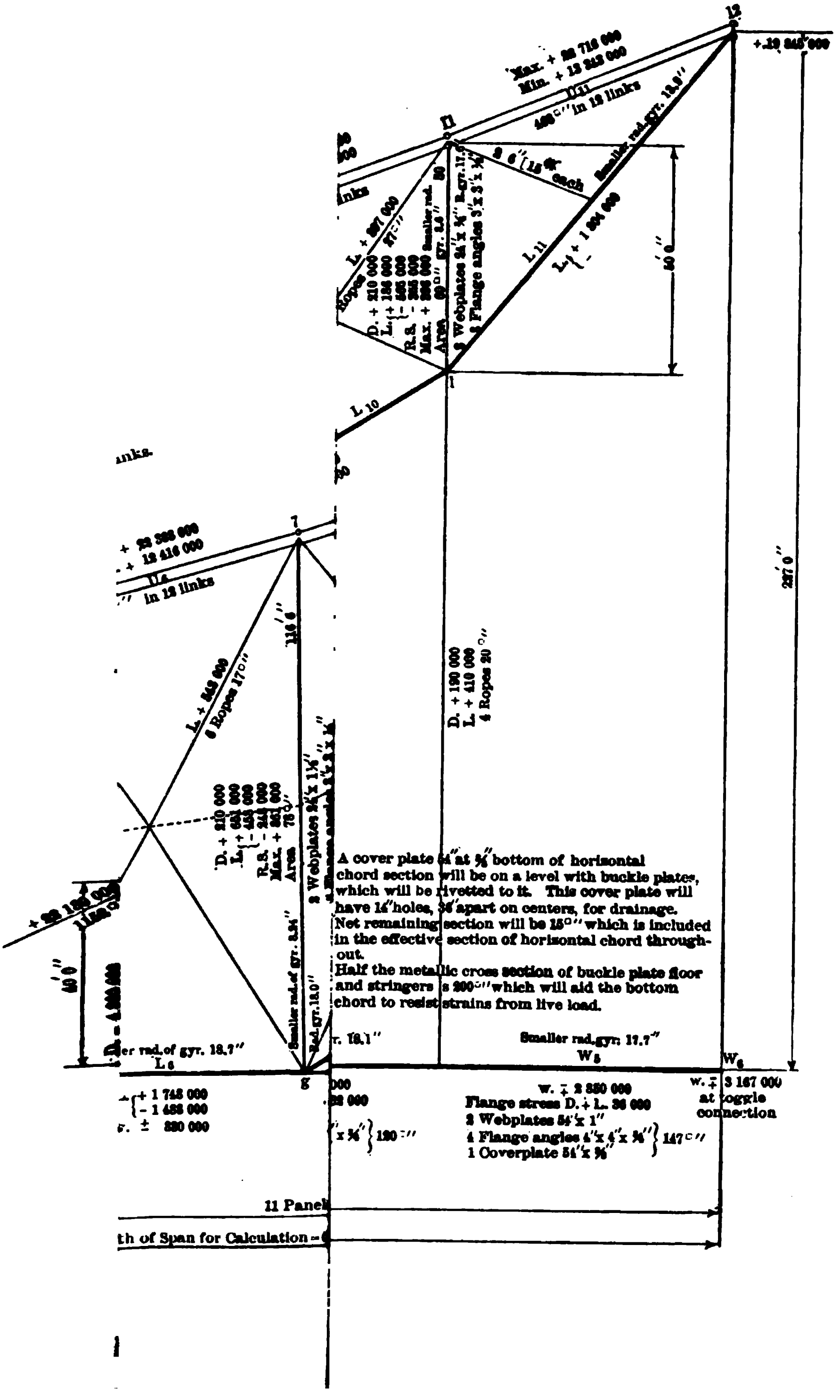
From a scientific point of view, the stiffening truss is a crude device, as compared with other modes of stiffening, which do not permit of the same laxity in dimensioning.

This appears most strikingly if that method of stiffening be applied to an erect arch. Small arches of that kind, in combination with trusses, had been built of wood and cast iron many years ago, when the art of bridge designing was yet in its infancy. But no one would think of applying that system to erect arches at this day, because better and more effective systems of arch bracing have been worked out since. As every suspension bridge is theoretically an inverted arch bridge, so, analogically, all the different systems of bracing in erect arches are applicable to suspended arches, and any system of bracing considered poor and inefficient in an erect arch is so, also, when used in a suspended arch. The principal difference is that the erect arch is in unstable equilibrium and therefore requires strong lateral cross-bracing, to keep it in position, while in the suspended arch, little or no such bracing is required. The center line of gravity in the suspended arch is far below the points of support on the towers, and the structure is in stable equilibrium. To that condition many badly designed suspension bridges owe their life; it covers a multitude of sins against good engineering.

The merit and serviceableness of a bridge structure, other things being equal, can be gauged by no other criterion so reliably as by the degree of its rigidity. The theory of cumulative vibration might find other fruitful fields of application. It has none in a bridge, dimensioned to present standards of safety. Absence of sufficient rigidity marks the inferior bridge. This, for instance, is one of the reasons why, for smaller spans, stone bridges, where practicable, will always be preferred to metal bridges, in spite of their greater cost. High metal trusses, the American practice, are preferred to low trusses, the English practice. Not only are such trusses stiffer, but also cheaper, a point which found its confirmation among others in the competition for the Atbara Bridge.

The same considerations apply to arches, erect and suspended. The points of largest disturbance are at the quarters of the main span,

PLATE XXXII.
PAPERS, AM. SOC. C. E.
AUGUST, 1904.
LINDENTHAL ON
STIFFENED SUSPENSION BRIDGES



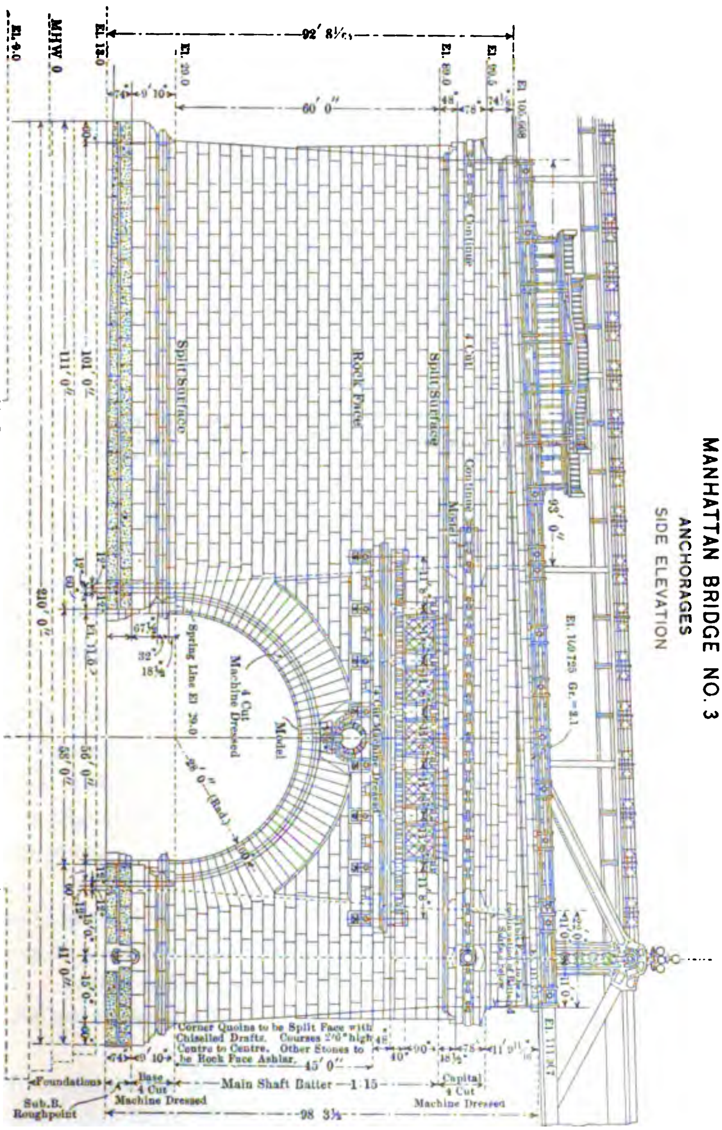

MANHATTAN BRIDGE NO. 3
ANCHORAGES
SIDE ELEVATION
Main Shaft Batter—1:15
Rock Face
Split Surface
Foundations
Base 4 Cut Machine Dressed
Capital 4 Cut Machine Dressed
Corner Quoins to be Split Face with Chiselled Drafts. Courses 2'0" high Centre to Centre. Other Stones to be Rock Face Ashlar.

Fig. 2.

and at the middle of the end spans. At those points the maximum bending moments from live load are greatest. The height of the stiffening frame at those points determines the rigidity of the arch, similarly as does in an ordinary truss its height at the middle.

Therefore, a form of stiffening frame having its greatest height at the quarters in the main span, and at the center of the side spans, will offer greater rigidity and also greater economy than any other; and, when attached directly to the arch, there will be the further economy of the saving of an entire chord.

Let us now bear in mind the familiar fact, that when the height of the bracing at points of greatest flexure attached to the cable (or arch) is equal to or greater than one-quarter of the sag of the cable, no additional material will be required for the cable sections for the bending strains from a one-sided live load.

We have then a combination of frame and arch or cable, resulting (similarly as in a truss) at once in great stiffness and great economy, which is an evidence of good design.

The suspension structures shown in Fig. 1 belong to that class, embodying the above-named features. The bottom chord is farthest from the cable at the quarters of the middle span and at the center of the side spans, and it coincides with the line of the floor for the middle half of the main span and for one-half of the side spans, where it can be utilized also for the wind trusses.

In the end spans we have to meet the important fact that, if they be half the length of the main span, the sections of the stiffening frame for the same height will be twice as heavy as that for the middle span. Therefore, it will be advisable to make the height at the middle of the end span greater, if possible, than at the quarter of the middle span, or to shorten the end spans.

With none of the known systems of arch or cable bracing is such great height readily obtainable between the chords at points of greatest flexure without incurring waste of material at other points.

This system may be designed with or without a hinge at the center of the main span.

In the first case the stresses will be statically determinate. For this one advantage, however, there would be the great disadvantage of making the lower chord useless for a wind truss just at the point of largest bending strains from wind pressure.

MANHATTAN BRIDGE
OVER THE EAST RIVER AT NEW YORK CITY.

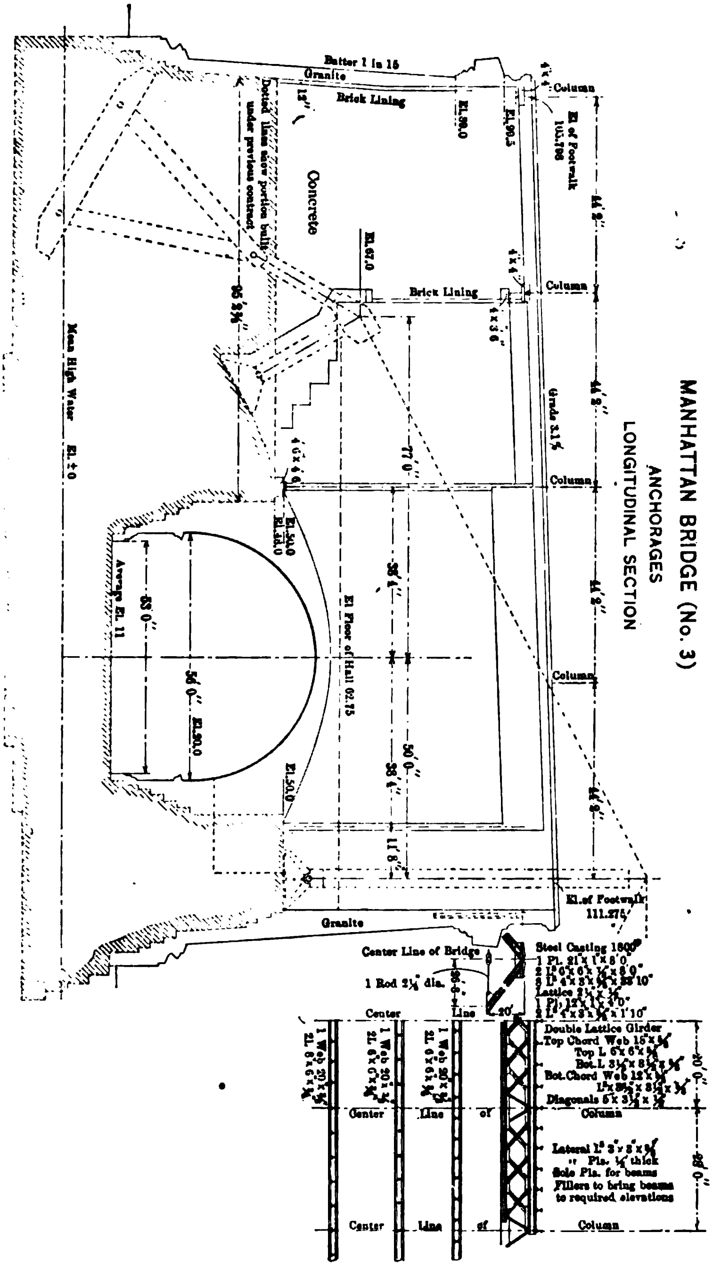

MANHATTAN BRIDGE (No. 3)
ANCHORAGES
LONGITUDINAL SECTION
Batter 1 in 15
Granite
Brick Lining
Concrete
Column
Mean High Water El. ± 0
Center Line of Bridge
Double Lattice Girder

Fig. 8.

The middle hinge has no decided advantage as regards temperature stresses. The writer has shown before,* that in stiffening trusses the middle hinge does not eliminate temperature effects, but that they are merely shifted.

The total amount of extra material for temperature stresses, in stiffening frames with parallel chords, is nearly the same in both cases. It is merely distributed differently. A similar rule holds good with other forms of stiffening, modified, however, in each case by the form of the frame. In order to keep the temperature stresses down, when there is no middle hinge, the frame should be made as low as possible along the middle third of the center span.

In Fig. 1, the upper figure represents the form proposed by the writer for the bridge over the St. Lawrence River at Quebec.

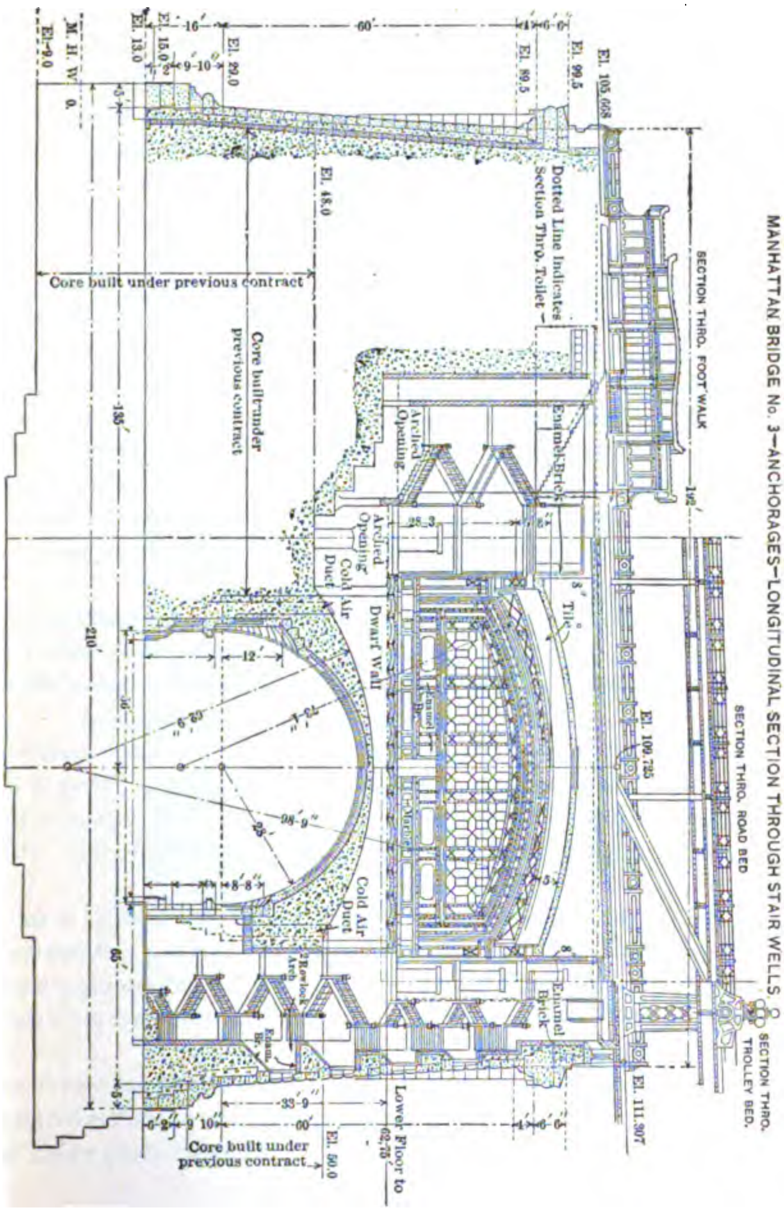
At the center of the middle span, the stiffening is 40 ft. high, or one-forty-fifth of the span (1 800 ft.). At the quarters, the stiffening frame is 110 ft. high, which is nearly one-sixteenth of the span, or six-tenths of the sag or deflection of the cable. A stiffening truss with parallel chords of equivalent rigidity under partial loads would require to be more than 100 ft. high, and its upper chord is shown by a dotted line.

That economy of material is not synonymous with economy of cost, found a new illustration in the competition for the Quebec Bridge. On the basis of the same specifications and unit stresses, the writer's suspension bridge was lighter in weight of metal, and in the amount of masonry no greater, than any of the competitive designs. But, nevertheless, the high prices for wire work made it more expensive than the cantilever design, which was adopted and is now in process of construction.

At the writer's request, Professor Melan afterward made an independent calculation of the stresses, sections and deflections, with the result that no changes were required in the sections, as shown on the strain sheet, Plate XXXII.

It should be remarked that in this system it is preferable that the cable assume its proper equilibrium curve under full dead load, and that the diagonals, which are without strain at a middle temperature, be made adjustable in length. The diagonals are strained only from live load and from temperature changes.

* *Engineering News*, March 10th, 1888; Appendix D of the Report of Engineers, published in *Engineering News*, November 22d, 1894.



MANHATTAN BRIDGE No. 3—ANCHORAGES—LONGITUDINAL SECTION THROUGH STAIR WELLS.
SECTION THRO. FOOT WALK
SECTION THRO. ROAD BED
SECTION THRO. TROLLEY BED.
Core built under previous contract
Core built under previous contract
Core built under previous contract
Dotted Line Indicates Section Thro. Toilet
Enamel Brick
Arched Opening
Arched Opening
Cold Air Duct
Cold Air Duct
Dwarf Wall
Tile
Rowlock Arch
Enamel Brick
Lower Floor to
M. H. W.
El. 105.668
El. 111.307
El. 109.735
El. 99.5
El. 89.5
El. 48.0
El. 50.0
El. 29.0
El. 15.0
El. 13.0
El. -9.0

FIG. 4.

If a settlement should take place in the foundation of the towers, any irregularity resulting therefrom, in the stresses of the stiffening frames, is readily corrected by a readjustment of the diagonals.

The pivotal form of tower simplifies greatly the details of cable bearings, avoids eccentric pressure, and facilitates the erection of cables. These great advantages have been recognized also in the recent designs for suspension bridges abroad; thus, the towers of the new Buda Pest Chain Bridge and of the proposed suspension structure over the Rhine at Cologne are both pivoted at the bottom.

The lower design on Fig. 1 represents the form of the Manhattan Bridge of New York on the same scale as that of the Quebec Bridge. At the center of the middle span, the stiffening frame is 22.15 ft. high, or one-sixty-sixth of the span (1 470 ft.). At the quarters, the height is 58 ft., which is about one-twenty-fifth of the span, and more than one-third of the sag or deflection of the chains in the main span.

A stiffening truss with parallel chords of equivalent rigidity under partial loads would be more than 55 ft. high, as shown by the dotted line. The trusses in the Williamsburg Bridge, 1 600 ft. span, are only 40 ft. high, equal to one-fortieth of the span.

The unsightliness of high stiffening trusses is not the only objection. Their greater weight and greater cost, and the greater temperature stresses in them, require no special proof. Strain sheets based on the same conditions are more reliable than general formulas.

In this paper the writer proposes to give merely the salient features of a type in the evolution of the rigid suspension bridge. That it is readily adapted to very satisfactory architectural treatment may be observed from the perspective view of the Manhattan Bridge. The two tower piers are completed.

For a better understanding of the plans of this bridge, it may suffice to state, briefly, that the main span and the two symmetrical side spans are 1 470 and 725 ft. long, respectively, and are supported from two steel towers resting on comparatively low masonry piers and rising to a height of 400 ft. above high water.

The superstructure of the main spans is suspended from four lines of eye-bar chains with fixed connections at the tops of the towers and at the shore anchorages. The eye-bars are of nickel-steel, which is 50% stronger than ordinary structural steel.

The advantages of pin connections with the bracing and suspend-

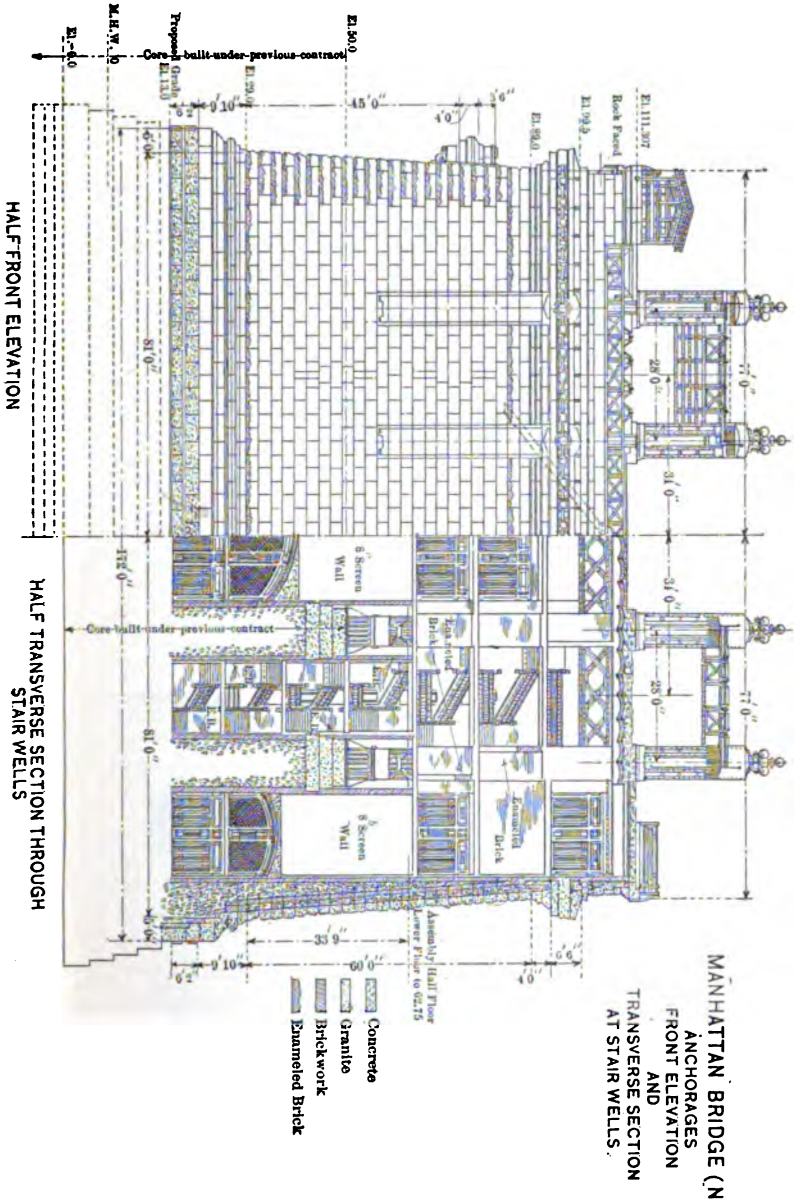
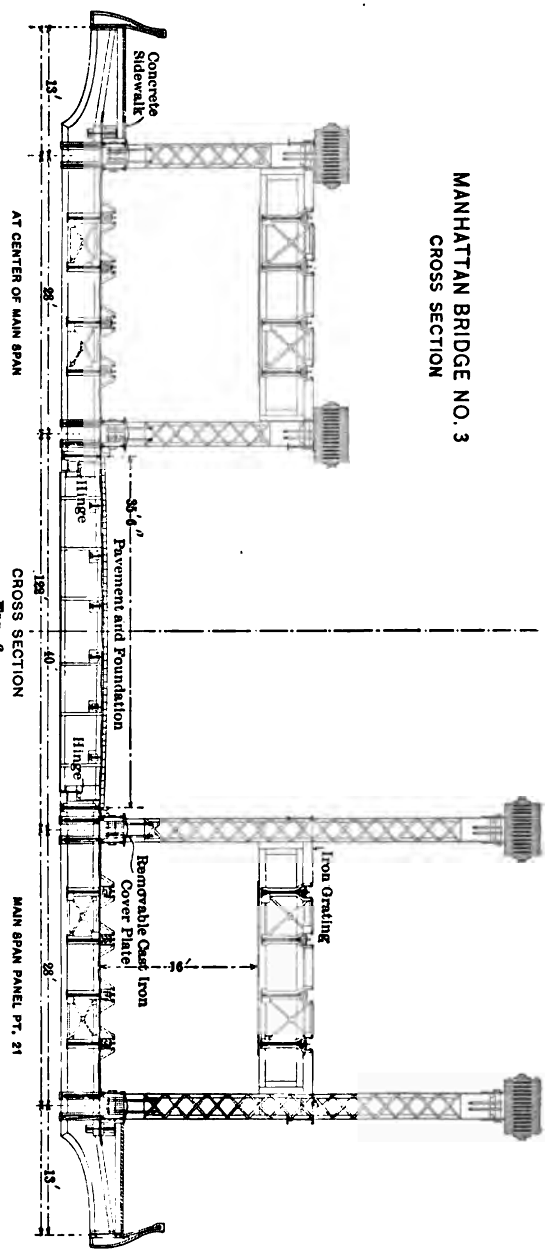

MANHATTAN BRIDGE (No.3)
ANCHORAGES
FRONT ELEVATION
AND
TRANSVERSE SECTION
AT STAIR WELLS.
HALF FRONT ELEVATION
HALF TRANSVERSE SECTION THROUGH
STAIR WELLS
Core built under previous contract
Concrete
Granite
Brickwork
Enameled Brick

Fig. 5



CROSS SECTION

Fig. 6.

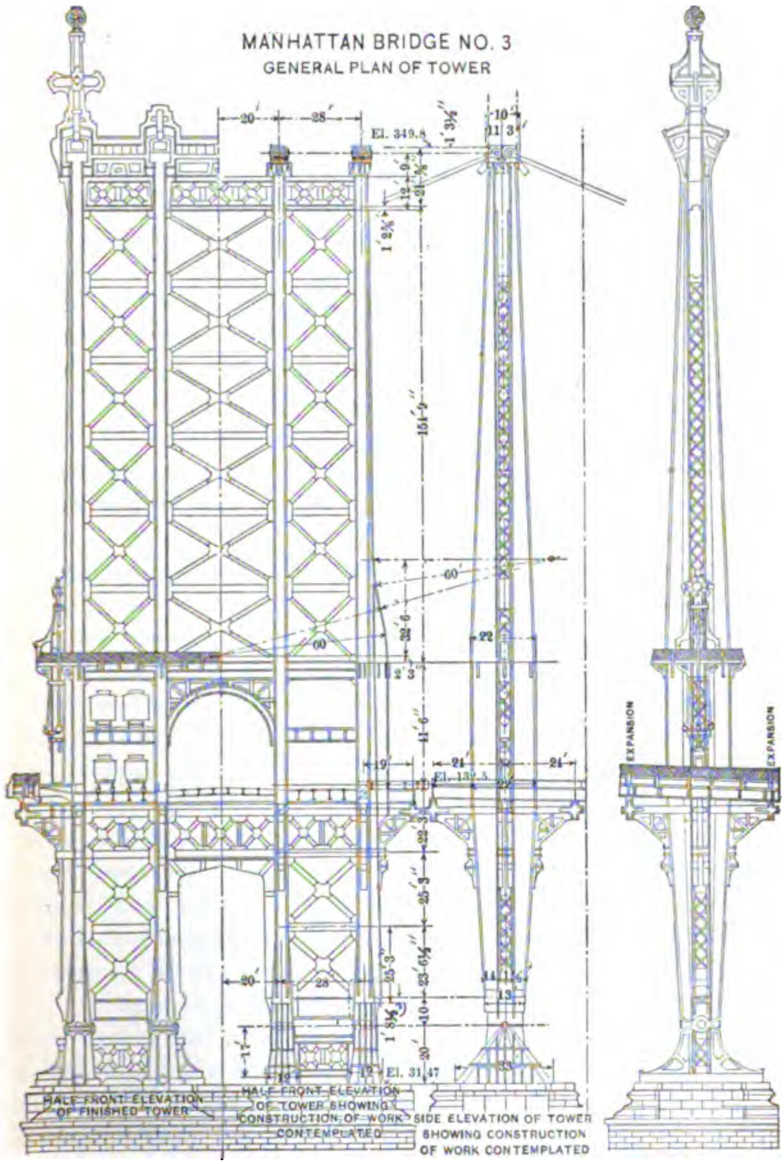

MANHATTAN BRIDGE NO. 3
GENERAL PLAN OF TOWER
El. 349.8
El. 132.5
El. 31.47
EXPANSION
EXPANSION
HALF FRONT ELEVATION OF FINISHED TOWER
HALF FRONT ELEVATION OF TOWER SHOWING CONSTRUCTION OF WORK CONTEMPLATED
SIDE ELEVATION OF TOWER SHOWING CONSTRUCTION OF WORK CONTEMPLATED

Fig. 7.

ers, and of manufacturing and erecting the superstructure as a connected whole are too obvious for extended description.

The main chains lie in vertical planes, 20 and 48 ft. from the axis of the bridge, and, at the shore ends, deflect from vertical steel posts into the heavy anchorage masonry.

The platform for the lower deck rests on plate-girder floor beams supported from the suspension chains. These floor beams are hinged at the interior chains, so that the loading of one side of the bridge will not affect the chains on the other side, and all ambiguity in the distribution of load is thereby prevented.

Each tower consists of a single transverse bent of four massive columns on the center lines of the chains. The four columns are vertical, and are thoroughly united by heavy transverse girders and by rigid cross-bracing.

The posts taper up and down from the roadway level in the plane of the chains. The lower end of each post has a pin bearing on a cast-steel shoe, which widens out to a broad base to distribute the pressure on the masonry.

The tower columns can be erected without staging, in the same manner as high chimneys are built.

During erection, wedges are inserted at the bottom of the tower columns, on a line with the pin bearings, as shown on the drawing. By means of these wedges, the verticality of the posts during erection can be readily secured. The wedge details are proportioned to resist a high wind pressure upon the tower columns during erection; after erection the wedges are removed.

Each anchorage is provided with a large transverse arch to allow street traffic through it. The interior cavities in the upper part of each anchorage are utilized for large auditorium halls, comfort stations and shelter rooms, to be accessible by elevators and stairways.

Like the tower piers, the anchorages are to be of concrete with ashlar facing, and will have heavy cornices and mouldings.

The aim has been to combine in the design of both superstructure and substructure simplicity of construction with architectural beauty of outline and details. The structure will be fire-proof throughout. The roadways will have solid buckle-plate floors. The design was not treated as for a mere utility structure, to be handed over to some decorator for adding the usual senseless ornaments, but as a truly artistic work of the engineer, working together with the architect in the study of form and expression of purpose.

The rigidity of the structure obtainable with this simple method of stiffening is most remarkable, as may be judged from the computed deflection of 22 in. (one-eight hundredth), at the center of the middle span, when this is fully loaded with 4 000 lb. emergency live load per lin. ft. of each chain, and both side spans unloaded.

The plans for this bridge have been passed upon by a board of bridge engineers appointed by ex-Mayor Low, and, from an æsthetic point of view, they have also been approved by the Municipal Art Commission of New York City.

Vol. XXX. AUGUST, 1904. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THEORY AND FORMULAS FOR THE ANALYTICAL COMPUTATION OF A THREE-SPAN SUSPENSION BRIDGE WITH BRACED CABLE.

By Leon S. Moisseiff, Assoc. M. Am. Soc. C. E.

To be Presented September 21st, 1904.

The scarcity of published information on fully-worked-out, practical applications of the theory of suspension bridges to long-span structures is well known to engineers. The writer hopes that the present paper will furnish some of the desired information, and will help to fill part of the gap in engineering literature, as far as it applies to bridges of the type described.

When, in the course of the writer's professional duties, the computation of a three-span suspension bridge with braced cables came up, he could not find, in engineering literature, the theory of this type developed in any way. All there was on hand for the problem consisted of the general principles of the elastic theory. The deduction of the required equations was made still more troublesome by the fact that the outlines of the stiffening trusses, as shown in Fig. 1, do

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not follow continuous curves convenient for integrations, and summations had to be used instead.

It is the writer's hope that the theory and formulas deduced in this paper are presented with sufficient clearness to enable the reader to follow their development without having to resort to mathematical textbooks.

These formulas were applied to the computation of a first-class city bridge, of long span, and of a capacity for city traffic greater than that of any already constructed for the same city. While the design of bridges of this type presents many interesting features, the writer does not believe the present time to be ripe for discussion, and will not treat of the merits and demerits of the bridge in general. Therefore, the theory alone will be presented, and only as many of the results as may be of general interest and may help to illuminate the subject.

In considering the design of a long-span suspension bridge, it is well to remember the main features by which it differs from bridges of other types. The suspension bridge, together with its antipode, the arch, produces on its abutments reactions the lines of action of which, unlike those in common beam and truss bridges, are not vertical, but, generally, more or less inclined to the horizontal. This, as is well known, is the distinctive characteristic of arch and suspension bridges, and it determines their design, construction, and behavior under load, and under the action of other causes. In fact, the existence of the horizontal components of the reactions is the great reason for the use of these bridges. While the com-

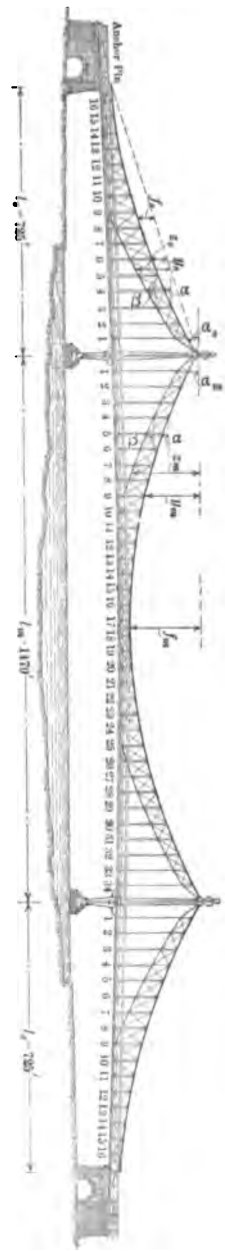


FIG. 1

mon beam and truss are in a sense self-contained, and their reactions on the abutments are independent of the geometric form of the truss, the material of which they are made and its elastic behavior, arch and suspension bridges are not self-contained, and their reactions are dependent on all these conditions. To make them self-contained, an additional member would be required, which would act as a tie between the supports, in the case of the arch, and as a strut, in the case of the suspension bridge. As it is, the rigidity of the earth supplies the required connection. The great importance of a slip in the abutments can readily be seen: It is equivalent to an undue change in length of the imaginary connecting member.

This imaginary connecting member would take up the horizontal components of the reactions; and the vertical components, only, would be transmitted to the abutments. The so-formed truss could then be computed practically as a simple beam. Actually, the function of this imaginary member is performed by the horizontal component of the reaction supplied by the abutments or anchorages.

As long as vertical forces, only, are acting on an upright or inverted polygon, it is evident, from the first principles of statics, that the horizontal component of the stress in the polygon will be constant, from point to point throughout the span, for the given system of forces. Naturally, the horizontal components of the reactions will then be equal to each other and to the constant component of the stress. This is a well-known property of the arch, and is common to all its possible geometric figures. It forms the very foundation of all rational arch theories. When confused by mathematical symbols and unknown variables, it is the constancy of the horizontal component which guides the engineer to a knowledge of moments and shears.

This is true for arches as well as suspension bridges, and, in so far, the theory of the arch applies also to its inverted form. In fact, the theory of these structures is practically the same, and to understand one means to know the other. But the computations based on their theory vary considerably because of considerations of an engineering and practical character.

The upright arch has its supports below its center of gravity, and, therefore, is in unstable equilibrium. The suspension bridge, on the contrary, has its supports above the center of gravity, and is in stable equilibrium.

If, on an upright arch of a given span and rise, a given system of loads is acting, and if the arch is assumed to consist of a number of members connected by frictionless hinges, there is only one geometrical polygon which will correspond to the requirements of equilibrium. This is its equilibrium polygon. If by any cause this polygon be disturbed from its original position, it will not tend to come back, but will depart from it and collapse. If the same arch is inverted, its equilibrium polygon will be identical with the former, but also inverted. But an important change has taken place meantime; when similarly disturbed, the inverted polygon will go back to its original position. To keep a flexible upright polygon in position would require great care in its design, and the provision of a number of outside forces to prevent its displacement. On the contrary, the inverted polygon must only be able to resist the stresses caused in it, and the force of gravity will guide it safely to its final position. The effect of the above properties on the erection of arches and suspension bridges is self-evident. All that is necessary, from this point of view, in the erection of the main member of a suspension bridge is to suspend it so that it will have the desired rise or versine when under a certain load. This enables the engineer to suspend all the fixed load of the bridge from the cable, without requiring any stiffening to keep it in proper shape.

Convenient as the stability of a suspended polygon may be, for erection, and safe as it may be, this movability does not satisfy the requirements of bridge traffic. Traffic is dependent on tractive power and grades, and the limits of the movability and deflection of a bridge are determined by the steepness of the grades which its traffic can endure. To reduce the movability and deflection of a suspension bridge, its equilibrium polygon is "stiffened." This can be accomplished in several ways. The most common of these are: The stiffening of the original equilibrium polygon by attaching to it a truss, one chord of which is frequently formed by the polygon itself, and the suspension from the flexible polygon of a sufficiently rigid truss. The deflections and distortions of a cable or chain are caused by changes in temperature and by loads traveling over the bridge, and it is the function of the stiffening truss to limit these distortions.

As we see, the fixed or dead load does not enter directly as a cause of distortion from the curve assumed by the cable under the weight of the bridge. On the contrary, due to its stability of form, it tends to reduce the distortion produced by the other causes, and its beneficial influence is much felt in heavy and flexible bridges. Hence the stiffening truss must be designed primarily to resist the distorting effect which will be caused by a given moving load in the various positions which it may assume. In addition, the effect of changes in temperature must be included. The cable itself, as the main supporting member, need only be designed for the greatest total load which may come on it. If sufficiently strong for the latter, and resting on towers of proportionate strength and well anchored, the bridge is safe from danger, even should some of the stiffening members buckle or break. The knowledge of this is greatly inducive to the sound sleep of the engineer, but structures are not built to break, even without danger of complete failure, and, in important city bridges, the buckling of a stiffening chord or the breaking of a few suspenders may frighten the public unnecessarily.

At no stage of the design of an important suspension bridge is good judgment and a thorough knowledge of its functions required more than in determining the moving loads for which to provide and the unit stresses to be allowed for the stiffening trusses. The moving load being decided, the choice of the depth and form of truss and of the allowable unit stresses fixes the sectional areas and deflections which will follow necessarily from the assumed values.

To eliminate the effect of the fixed load on the stiffening truss, the cable is erected together with the truss suspended from it, the floor system, paving, etc., and at a certain temperature, assumed as normal, the truss is adjusted for its final connections. The fixed load on the bridge before the closing of the truss, therefore, cannot cause any stresses in the latter. Any additional load which may come on the bridge after this will causes stresses in the stiffening truss. So, also, will a change of temperature from the normal. From now on, the truss and the cable must act together; they form one elastic system. The action of this system will be dependent on the geometrical configurations of the component parts and on their elastic behavior. This will be seen plainly in the equations which will be developed in the following:

As stated before, the three-span bridge, the theory of which is presented here, is of the braced-cable type, and of the outlines shown in Fig. 1. The upper curve represents the main cable, and follows the equilibrium polygon of the weight of the bridge. It also forms the upper chord of the stiffening truss, which is attached directly to it. Under the action of the fixed load, and at an assumed normal temperature, there will be no stresses in the members of the stiffening truss. Any additional fixed or moving load which may come on the bridge, or any departure from the normal temperature, will cause stresses in the truss, which will then act as an inverted, braced arch over three spans. As shown in Fig. 1, the ends of the main- and side- span trusses come on pins or hinges, while they are continuous between these points. It is obvious that they form three two-hinged arches, and, as such, are statically indeterminate in the first degree. This means that the equations furnished by the conditions of static equilibrium are not sufficient to determine the stresses, and that one more equation of condition is required. It is furnished by the conditions of the elastic behavior of the truss. In one of its forms it is well known to engineers under the name of the "Principle of Least Work," and has been discussed repeatedly in the *Transactions* of this Society.

Algebraically, the general expression for the principle of least work is written:

$$\frac{\delta W}{\delta X} = \int \frac{P}{E A} \frac{\delta P}{\delta X} d s + \int \frac{M}{E I} \frac{\delta M}{\delta X} d s = 0 \ldots\ldots\ldots (1)$$

in which,

W = the total elastic work in the system when free from vibration;
X = an unknown force, assumed as an independent variable;
P = the direct axial force acting in any part of the system;
M = the bending moment acting in any part of the system;
s = the length of the member considered;
E = its coefficient of elasticity;
A = its cross-sectional area;
I = its moment of inertia about its neutral axis.

It has generally been found convenient to take the horizontal component of the pull in the cable as the unknown variable indicated by X in Equation 1. As stated before, the horizontal component, which may be denoted by H, represents the characteristic mark of the arch or suspension bridge. It forms a function of the stress in any part of the structure, and is constant at any section for a given condition of loading. These properties make it convenient to express the stress and work of the several parts in terms of H, as was done in the case described.

The general method used for the derivation of the formula for the horizontal pull is common to most applications of the least work principle, and is the same as developed by Professor J. Melan in his book on "Arches and Suspension Bridges." The deduction of the equations for a three-span suspension bridge of the type described is original with the writer.

After the lengths of the spans, the required clearances for the purposes of navigation, and the grades have been fixed, and the desirable versine of the cable has been chosen, the curve of the cable which forms the upper chord of the truss has thereby been determined. In the main span the unstiffened cable will naturally assume the form of the equilibrium polygon of the fixed load on it. As the latter is practically uniformly distributed over the whole of the span, its equilibrium polygon will approximate a parabola. For all preliminary computations, and even for most final computations, it has been found that the curve of the cable may be assumed to be parabolic.

The form of the curves of the side spans are determined by that of the main span, because the three spans must balance, under the fixed load and normal temperature. In other words, the horizontal component of the pull in the cable must be the same throughout the bridge. For a uniform load over the whole main span on the unstiffened cable, it is, as is well known:

$$H_1 = \frac{p_m \, l^2_m}{8 f_m} \qquad (2)$$

in which,

p_m = the load on the main span, per linear foot of bridge;
l_m = the length of the main span, from the centers of the towers; and
f_m = the versine of the main-span cable.

The expression for the horizontal pull in the side-span cable is, similarly:

$$H_2 = \frac{p_s \, l^2_s}{8 f_s} \qquad (3)$$

in which the suffix, s, indicates the similar notation for the side spans. As stated in the foregoing, the balancing of the three spans requires that $H_1 = H_2$, or

$$\frac{p_m \, l^2_m}{f_m} = \frac{p_s \, l^2_s}{f_s} \qquad (4)$$

Whence the versine of the side-span parabola:

$$f_s = \frac{p_s\, l^2_s}{p_m\, l^2_m} f_m \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

As can be seen from this expression, the value of f_s is determined by the data.

If the uniform weight of the bridge, including the cables, is the same for the main and side spans, $p_m = p_s$, the expression for the side-span versine becomes:

$$f_s = \frac{l^2_s}{l^2_m} f_m \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Or, in other words, the versines are to each other as the squares of their spans. For all preliminary computations, the use of this relation is perfectly justified. If the fixed load is not uniformly distributed, the versines will be to each other as the greatest moments due to the loads on a simple beam. It should be noted, however, that the versine of the side spans, as well as all curve ordinates, must be measured at the center of the span from the straight line drawn through the center of the pin at the top of the tower to the center of the end pin at the anchorage as indicated in Fig. 1.

It will be noticed that the versine of the side-span curve, f_s, is independent of the elevation of the anchor pin relative to the tower. The latter elevation is determined by considerations as to the steepest grade allowable and the least cost of the anchorage masonry.

The curves of the cable having been determined and drawn, the outlines of the stiffening trusses are laid out. The form of the lower chord, the depths of the truss and the panel arrangement are governed by considerations of economy and good looks. The type of the truss discussed in this paper is adaptable to such depths of truss as will generally follow the growth of the bending moments.

The floor system can now be designed and its weight computed, together with the weight of railings, pavement, rails, etc. In the present case, which is that of a long-span city bridge with solid buckle-plate floor, the weight of the foregoing items constituted one-half of the total fixed load on the bridge. With a thorough understanding of the influences and effects of the several parts of the structure, it is now comparatively easy to determine approximately the fixed load on the bridge, the probable areas of the cable and stiffening chord at the center of the main span, the average areas of the cable and stiffening chord, and the approximate moments of inertia at any panel point.

At normal temperature and under fixed load, only, the stiffening chord will, by the assumptions of the design and erection, be free of stress. With the advance of an additional load, the trusses will act as inverted arches, and the load will be distributed between cable and stiffening chord. For purposes of computation, it is important to determine approximately the relative share of the load sustained by each of them. The following formula has been found to give closely approximate results.

If the two chords of the truss are assumed to be suspended without any diagonal stiffening members between them, a load on the bridge would be sustained by the two chords in proportion to their vertical deflection under the stress caused. Let us denote by H the sum of the horizontal components in both chords and by $m\,H$ and $n\,H$ the proportion of H taken up by the cable and by the lower chord, respectively; so that,

$$H = m\,H + n\,H, \text{ and } m + n = 1 \quad \text{(7)}$$

If, further, A_1 denotes the average area of the cable in the span considered, A_2 the average area of the stiffening chord, L_1 the length of the cable, L_2 that of the chord, f_1 the versine of the cable and f_2 that of the chord, the following relation will hold true:

$$\frac{H_1}{H_2} = \frac{A_1\,L_2\,f_2}{A_2\,L_1\,f_1} = \frac{m}{n} \quad \text{(8)}$$

The Determination of the Horizontal Pull in Cable and Chord Due to Moving Load.—

Let

H = the total horizontal pull in the cable and stiffening chord;
$m\,H$ = " horizontal pull in the main-span cable;
$n\,H$ = " " " main-span stiffening chord;
$m_1\,H$ = " " " side-span cables;
$n_1\,H$ = " " " " stiffening chord;
A_c = " cross-sectional area of the cable at any point;
C_m = " " " main-span stiffening chord;
C_s = " " " side-span " "
I = " moment of inertia of the truss at any point;
E = " coefficient of elasticity of the material;
h = " height of the tower;

y_m and y_s = the ordinates of the cable, in main and side spans, respectively;

z_m and z_s = " ordinates of the stiffening chord, in main and side spans;

α = " angle the cable makes with the horizontal at any point;

α and α_s = " angles it makes with the horizontal at the tower pin, in main and side spans, respectively;

β = " angle the stiffening chord makes with the horizontal at any point;

s = " length of the cable curve;

a = " panel length.

The bridge is symmetrical about an axis through the center of the main span.

To express the stress in the several parts of the structure in terms of the unknown horizontal pull, H, is quite simple. The portion of H acting in the main cable is $m\ H$, and, since the stress in the cable varies as the secant of its angle, α, with the horizontal, it is:

$$m\ H \sec.\ \alpha = m\ H \frac{d\ s}{d\ x}.$$

Similarly, for the side spans:

$$m_1\ H \frac{d\ s_1}{d\ x}.$$

The direct stress in the stiffening chords as suspended members, similarly to that in the cables, is expressed by:

$$n\ H \sec.\ \beta, \text{ for the main span,}$$

and

$$n_1\ H \sec.\ \beta, \text{ for the side spans.}$$

The stress caused in the stiffening truss, as such, by bending moments due to moving loads is somewhat less simple. However, the writer believes that the action of the stiffening truss is explained clearly by the simple algebraic form in which the bending moment at any section of the truss can be expressed.

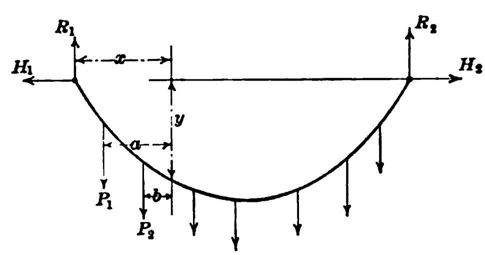


FIG. 2.

Let Fig. 2 represent

an inverted rigid arch of any form which is subjected to the action of a number of vertical forces, P_1, P_2, etc.

The reaction at the points of support can be decomposed into the vertical reaction, R_1, and the horizontal pull, H. As the external forces on the arch are all vertical, they have no horizontal component. The horizontal reaction, H, therefore, is the same at both points of support, and the horizontal component of the direct stress in the arch is constant throughout for the given system of loads. The vertical component, on the contrary, varies from the point of application of each load; and the vertical reactions, R_1 and R_2, follow the law of the lever, as for simple beams. If we now take moments about any point, x, y, we get:

$$M_x = R_1 x - (P_1 a + P_2 b + \quad . \quad . \quad . \quad) - Hy \dots\dots \quad (9)$$

But it will be observed that the first two expressions on the right side represent the bending moment of a simple beam resting freely on its supports. If we denote this bending moment by $\mathfrak{m}$ we may write:

$$M_x = \mathfrak{m} - H y \dots\dots\dots\dots\dots\dots \quad (10)$$

This is the general expression for the bending moment at any point, for a suspension bridge, as well as for an upright arch. In plain words, it states that, as far as bending is concerned, both the upright and the inverted arch act as simple beams which are relieved by a moment, $H y$, which is due to the horizontal reaction, H, acting at the distance, y, from the point, x, y. It is well to remember the general form and meaning of this expression for the moment, because it represents the fundamental idea of the arch, and, in the most complicated cases, it will serve as an excellent method of orientation.

For the truss discussed in this paper, the general form of the moment, expresed in H, becomes:

For the main span, $M_x = \mathfrak{m} - m H y_m - n H z_m \dots .$ (11)

" " side spans, $M_x = \mathfrak{m} - m_1 H y_s - n_1 H z_s \dots .$ (12)

The general expression for the principle of least work, Equation 1, is:

$$\frac{\delta W}{\delta H} = \int \frac{P}{E A} \frac{\delta P}{\delta H} d s + \int \frac{M}{E I} \frac{\delta M}{\delta H} d s = 0.$$

Since the coefficient of elasticity will be practically the same for all parts of the structure, it may be omitted altogether. Table No. 1, Plate XXXIV, contains the tabulation of the foregoing expression extended over the several parts of the system.

$$\int \frac{P}{A} \frac{\delta P}{\delta H} ds + \int \frac{M}{I} \frac{\delta M}{\delta H} ds = 0.$$

Cable, m A_c **the area at the center be approximated and the cable be assumed to section as its secant, we get for parabolic curves:**

$$\frac{m^2}{A_c} l_m \left(1 + \frac{16}{8} \frac{f^2_m}{l^2_m}\right) H \text{ and } 2 \frac{m^2_1}{A_c} l_s \left(1 + \frac{16 f^2_s}{8 l^2_s}\right) H.$$

Cable, si

Stiffening

Stiffening

Stiffening $2 m n \sum \frac{y_m z_m a}{I} \Big] H - m \sum \frac{\mathfrak{m}\, y_m a}{I} - n \sum \frac{\mathfrak{m}\, z_m a}{I}$

Stiffening $\frac{a}{} + 4 m_1 n_1 \sum \frac{y_s z_s a}{I} \Big] H - 2 m_1 \sum \frac{\mathfrak{m}\, y_s a}{I} - 2 n_1 \sum \frac{\mathfrak{m}\, z_s a}{I}$

$$H = \frac{\frac{a}{}}{\left[m \frac{a}{} + 2 m^2_1 \sum \frac{y^2_s a}{I} + 2 n^2_1 \sum \frac{z^2_s a}{I} + 4 m_1 n_1 \sum \frac{y_s z_s a}{I} \right]} \quad \text{.. (13)}$$

$$= D$$

It will be noticed that the work of the web members of the truss is not included in the summation of Table No. 1. The effect of these members is very small, and, since they had to be made adjustable, it did not appear necessary to include them.

A glance at the denominator of the expression deduced for H, in Equation 13, which we will denote by D, shows that it is independent of any load on the bridge and dependent on such values only as form the geometric and elastic characteristics of the structure. For a given structure, of the sections and dimensions assumed, the denominator, D, is constant, whatever the load on the bridge. Therefore, it need be computed only once, which reduces greatly the labor of computation.

From the form of the equation,

$$H = \frac{1}{D}\left[m \sum \frac{\mathfrak{m}\, y_m\, a}{I} + n \sum \frac{\mathfrak{m}\, z_m\, a}{I} + 2\, m_1 \sum \frac{\mathfrak{m}\, y_s\, a}{I} + 2\, n_1 \sum \frac{\mathfrak{m}\, z_s\, a}{I}\right] \ldots\ldots\ldots\ldots (14)$$

it is seen that the expressions in the numerator which represent the effect of the load can be treated independently; the first expression in the brackets being the effect of moving load on the main span, and the second that on the side spans. The coefficients, 2, in the latter are indicative only of the existence of two symmetrical side spans. In the case of an identical load on both side spans, the coefficients would represent the actual effect of the load on H.

The expressions contained in Table No. 1 are general, and true for any form of cable curve and variable panel strength, and the summations may be much simplified for regular curves and equal panel lengths.

H for Uniform Load from Anchorage to Anchorage.—To obtain the greatest reactions on the towers and at the anchorages, the bridge must be loaded with its full uniform load from anchorage to anchorage. For a uniform load, p, per unit of length, the simple beam moment:

$$\mathfrak{m} = \frac{p}{2}\, x\, (l - x) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in which l indicates the length of span. For a parabolic curve of the cable, the ordinate of the parabola,

$$y = \frac{4 f}{l^2}\, x\, (l - x) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

for the origin of the co-ordinates at the center of the tower pin. f indicates the versine of the curve. Substituting these values for $\mathfrak{m}$ and x, it becomes:

$$m \sum \frac{\mathfrak{m}\, y_m\, a}{I} = m \frac{p\, l^2_m}{8\, f_m} \sum \frac{y^2_m\, a}{I},$$

$$n \sum \frac{\mathfrak{m}\, z_m\, a}{I} = n \frac{p\, l^2_m}{8\, f_m} \sum \frac{y\, z\, a}{I},$$

and similarly for the side spans. The total horizontal pull in the cable and chord, due to a full uniform load, p, per horizontal unit of length on the main span only, we then get as:

$$H_m = \frac{p\, l^2_m}{8\, f_m} \times \frac{m \sum \frac{y^2_m\, a}{I} + n \sum \frac{y\, z\, a}{I}}{D} \dots\dots\dots (17)$$

For a full load on one of the side spans:

$$H_s = \frac{p\, l^2_s}{8 f_s} \times \frac{m_1 \sum \frac{y^2_s\, a}{I} + n_1 \sum \frac{y_s\, z_s\, a}{I}}{D} \dots\dots\dots (18)$$

For the load extending from anchorage to anchorage, the effects of each span must be added, so that

$$H_{full.} = H_m + 2\, H_s \dots\dots\dots\dots\dots\dots\dots\dots (19)$$

To obtain the proportions of H sustained by cable and stiffening chord, the foregoing values of H are multiplied by the proportions, m and n, deduced before.

H for a Single Concentration.—If a single concentrated load, P, acts on a simple beam, of a span, l, at a distance, v, from the point of support, the expression for the bending moment due to it will take the form

$$\mathfrak{m}_{o-v} = \frac{P\,(l - v)\, x}{l} \dots\dots\dots\dots\dots\dots\dots (20)$$

for any section, x, between o and v, and

$$\mathfrak{m}_{v-l} = \frac{P\,(l - x)\, v}{l} \dots\dots\dots\dots\dots\dots\dots (20a)$$

for any section, x, between v and l.

Substituting these expressions for $\mathfrak{m}$ in the numerator of the general equation for H:

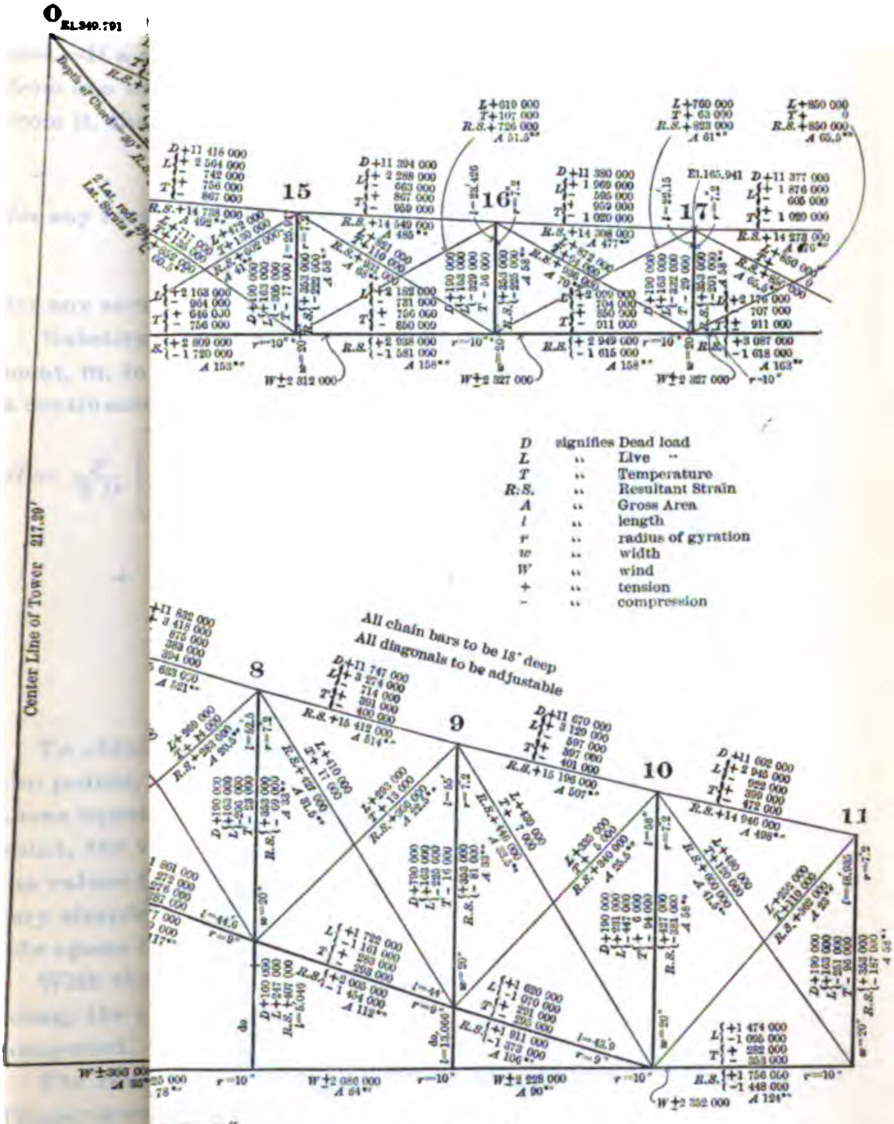
$$\sum \frac{\mathfrak{m}\, y\, a}{I} \text{ and } \sum \frac{\mathfrak{m}\, z\, a}{I},$$

we obtain for a concentrated load, P:

$$H_P = \frac{P}{D}\Bigg[m \sum_o^v \frac{a\, x\, y}{I} - \frac{m\, v}{l} \sum_o^l \frac{a\, x\, y}{I} + m\, v \sum_v^l \frac{a\, y}{I}$$
$$+ n \sum_o^v \frac{a\, x\, z}{I} - \frac{n\, v}{l} \sum_o^l \frac{a\, x\, z}{I} + n\, v \sum_v^l \frac{a\, z}{I} \Bigg] \dots\dots (21)$$

PLATE XXXV.
PAPERS, AM. SOC. C. E.
AUGUST, 1904.
MOISSEIFF ON
THREE-SPAN SUSPENSION BRIDGE.

4 trusses.



35 Panels. 42'each—1470'

This equation for H represents the analytical expression for the influence line of H for a load, P, traveling over the bridge. It is equally true for the main and for the side spans, for which, of course, the corresponding values and summations must be used.

H for a Uniform Load Continuous from Point of Support to any Section.—If a uniform load, p, per unit of length, advances continuously from one end of a simple beam, of a span, l, to any point distant u from it, the expression for the bending moment due to this load will be:

$$\mathfrak{M}_{o-u} = \frac{p\,u}{2\,l}(2\,l - u)\,x - \frac{p\,x^2}{2} \quad \text{...............} (22)$$

for any section between o and u, and

$$\mathfrak{M}_{u-l} = \frac{p\,u^2}{2\,l}(l - x) \quad \text{...... ....} (22a)$$

for any section between u and l.

Substituting these expressions for the simple-beam bending moment, $\mathfrak{M}$, in the numerator of the general equation for H, we get, for a continuously advancing load, p:

$$H = \frac{p}{2\,D}\left[m\,u\left(2 - \frac{u}{l}\right)\sum_{o}^{u}\frac{a\,x\,y}{I} - m\sum_{o}^{u}\frac{a\,y\,x^2}{I}\right.$$

$$+ \frac{m\,u^2}{l}\sum_{u}^{l}(l - x)\frac{a\,y}{I} + n\,u\left(2 - \frac{u}{l}\right)\sum_{o}^{u}\frac{a\,z\,x}{I}$$

$$\left. - n\sum_{o}^{u}\frac{a\,z\,x^2}{I} + \frac{n\,u^2}{l}\sum_{u}^{l}\frac{(l - n)\,a\,z}{I}\right] \quad \text{........} (23)$$

To obtain the value of H due to a partial uniform load between any two points, we need only subtract, from the value computed from the above equation for the load extending from the support to the farther point, the value computed due to the load to the nearer point. As the values for H are tabulated from panel point to panel point this is very simple. The equation deduced for H applies to either main or side spans for the corresponding values and summations.

With the several equations which have been established in the foregoing, the effect of any kind of moving load on the bridge can be computed.

The Determination of the Horizontal Pull in Cable and Chord Due to Temperature Changes.—The stiffening truss is free of stress at an assumed normal temperature which is the temperature of the final adjustment of the bridge under its fixed load only. Any change of temperature from the normal will cause stresses in the stiffening truss members. If ω denotes the coefficient of expansion of steel per 1° Fahr. and t the number of such degrees from the normal temperature which any member, of length, s, has departed, the change in length of this member will be $\omega\, t\, s$. If this member is under stress, P, a change in its length in the sense opposite to the stress will produce the work $P\, \omega\, t\, s$. The least-work expression for the effect of a uniform change of temperature will then be

$$\pm \int \omega\, t\, \frac{\delta P}{\delta H}\, d\, s,$$

which must be added to the summation of Table No. 1. Applying this expression to the several parts of the system, we get, by referring to Table No. 1:

Due to cables in main span: $m\, \omega\, t\, \Sigma_0^{lm}\, a \sec.^2\alpha$;

" " side span: $2\, m_1\, \omega\, t\, \Sigma_0^{ls}\, a \sec.^2\alpha$;

" stiffening chord in main span: $n\, \omega\, t\, \Sigma_0^{lm}\, a \sec.^2\beta$;

" " " side " $2\, n_1\, \omega\, t\, \Sigma_0^{ls}\, a \sec.^2\beta$.

It will be noticed that none of the above expressions is a function of H, and, therefore, they will appear in the numerator of the H equation, not disturbing the value of the denominator, D. The expression resulting for the effect of a change of t degrees in the temperature of the structure is:

$$H_t = \mp \frac{E\, \omega\, t}{D}\, [m\, \omega\, t\, \Sigma_0^{lm}\, a \sec.^2\alpha + 2\, m_1\, \omega\, t\, \Sigma_0^{ls}\, a \sec.^2\alpha + n\, \omega\, t\, \Sigma_0^{lm}\, a \sec.^2\beta + 2\, n_1\, \omega\, t\, \Sigma_0^{ls}\, a \sec.^2\beta] \quad \ldots .(24).$$

Moments and Stresses in the Side-Span Trusses.—It will be found, generally, that the side spans in three-span suspension bridges are too rigid to get the greatest bending moments, under the action of a partial moving load on them. The greatest positive moment, by which we will denote the moment giving a downward deflection, will be caused by the full moving load on the side span and no load on the remainder of the bridge. Going back to the general expression for moments, Equation 12,

$$M_x = \mathfrak{m} - m_1\, H\, y_s - n_1\, H\, z_s.$$

The value of H for a full load on one side span has been deduced in Equation 18, and the moment of a simple beam for a uniform load over the full length of the span,

$$\mathfrak{m} = \frac{p\, x\, (l_s - x)}{2}.$$

PLATE XXXVI.
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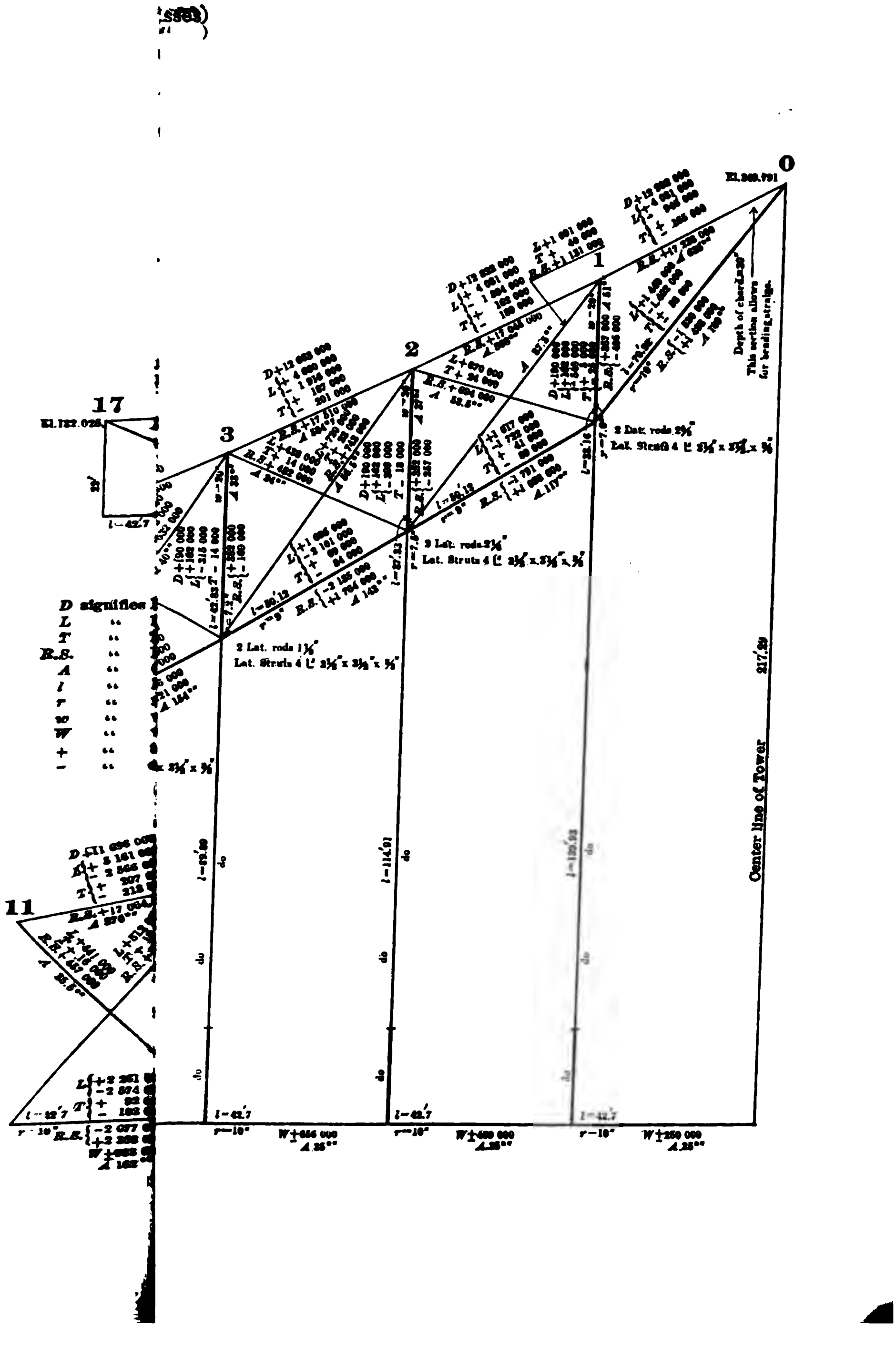
THREE-SPAN SUSPENSION BRIDGE.



The expression for the moment thus becomes:

$$\text{Maximum positive } M_x = \frac{p\,x\,(l_s - x)}{2} - m_1\,H\,y_s - n_1\,H\,z_s \ldots\ldots (25)$$

The greatest negative moments in the side span will be produced by the full load on the main span and far side span and no load on the span considered. It is given by

$$\text{Maximum negative } M_x = -\,m_1\,H\,y_s - n_1\,H\,z_s \ldots\ldots\ldots\ldots (26)$$

H represents here the value of the horizontal component due to full load on the main span and one side span. Equations 17 and 18.

The moments produced by a uniform change in temperature of t degrees either way follows at once from the general expression for moments:

$$\text{Temperature moment } \pm M = \mp\,m_1\,H_t\,y_s \mp n_1\,H_t\,z_s \ldots\ldots (27)$$

but it will be remembered that in Equation 24 H has a negative sign, so that the temperature moment for an increase in temperature will be:

$$\pm M = \pm\,m_1\,H_t\,y_s \pm n_1\,H_t\,z_s \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

After the moments have been computed for the several conditions of loading and temperature giving the greatest positive and negative moments, it is very simple to find the chord stresses in the stiffening truss produced by the bending moments, by dividing the latter by the corresponding lever arms. The chord stresses thus obtained must be added to the direct axial stress in the chord, which is given by the corresponding value of H multiplied by the secant of the angle which the chord makes with the horizontal. The sum of both represents the total stress in the chord at the given point. For the upper chord, of course, the stress due to the fixed load is included.

The stresses in the web members of the stiffening trusses due to the several conditions of load and temperature are determined by the method of moments, the same as for any truss with curved chords, except that the horizontal reaction, H, as well as the vertical reaction, must be considered.

Moments and Stresses in the Main-Span Trusses.—For the full uniform load on the main span and no load on the side spans, the bending moment will, similarly to Equation 25 deduced for the side span, be given by:

$$M_{tot.} = \frac{p}{2}\,x\,(l_m - x) - m\,H\,y - n\,H\,z \ldots\ldots\ldots\ldots (29)$$

H represents here the value of H for the full load on the main span, as given by Equation 17.

But in most sections, the greatest moments and stresses are produced by partial moving loads. The positions of loading giving the greatest positive and negative bending moments and shears have been determined graphically by the so-called "locus-line" or *Kaempferdrucklinie.*

If a concentrated load, P, travels along the span of an upright or inverted arch, the lines of action of the reactions produced at its ends will intersect in one point with the line of action of the load.

The curve generated by the continuous succession of this point as the load travels over the span, which is the locus of the points of intersection, may be denoted as the locus-line. The deductions, both analytical and graphical, of the thrust-line and its application to determine the positions of moving load for greatest chord and web stresses have been discussed fully in the works of Mueller-Breslau, Weyrauch and Melan.

The greatest negative moment at any point, due to partial load on the main span, is caused by the load extending from the far tower to the limiting section found for this point. As there is no load from the near tower to the limiting section, the value, $\mathfrak{m}$, of the general equation for moments represents the simple beam reaction, R_1, on the unloaded side, by the distance of the section from it. The equation for the greatest negative bending moment at x thus becomes:

$$M_{min.} = R_1\, x - m\, H\, y_m - n\, H\, z \ldots\ldots\ldots\ldots\ldots (30)$$

For a uniform load, p, per unit of length, extending from the limiting section, distant u from the left tower to the right tower, the moment may be written:

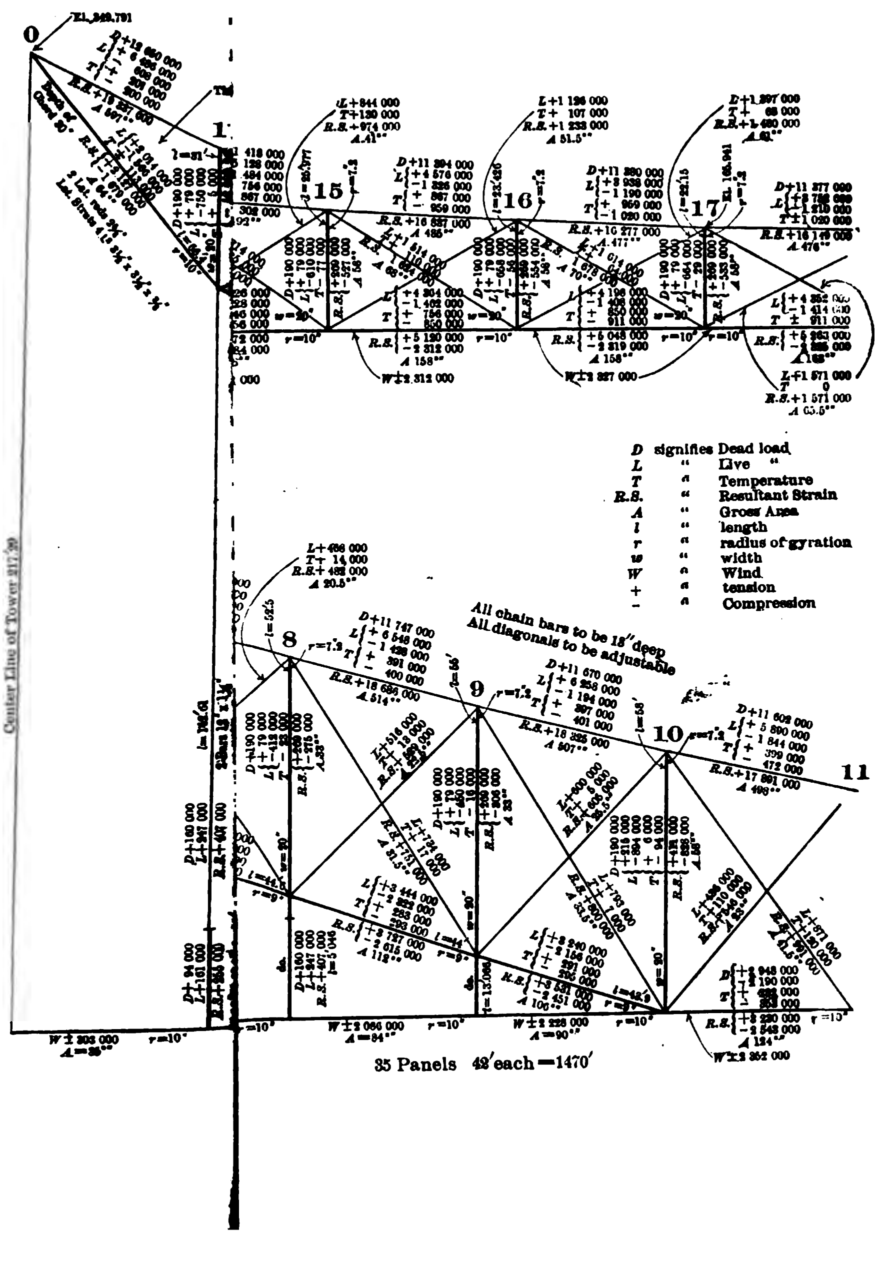
$$M_{min.} = \frac{p\,(l-u)^2\, x}{2\, l} - m\, H\, y_m - n\, H\, z_m. \ldots\ldots (31)$$

The values of H in this equation, of course, must represent the values corresponding to the loading used.

The greatest positive moments will be found by subtracting algebraically the corresponding values obtained from Equations 30 or 31, from those obtained from Equation 29:

$$M_{max.} = M_{tot.} - M_{min.} \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

The greatest negative moments on the main span which will be produced by loads on the side spans will take place with the latter fully loaded. If, as before, H_s (Equation 18), represents the total horizontal pull produced by the full load on one side span, the


D signifies Dead load
L " Live "
T " Temperature
R.S. " Resultant Strain
A " Gross Area
l " length
r " radius of gyration
w " width
W " Wind
+ " tension
− " Compression
All chain bars to be 18" deep
All diagonals to be adjustable
35 Panels 42' each = 1470'
Center Line of Tower 217.39

greatest negative moment due to both side spans being loaded will be:

$$M_s = -2\,m\,H_s\,y_m - 2\,n\,H_s\,z_n \ldots\ldots\ldots\ldots\ldots (33)$$

The two negative moments, Equations 31 and 33, must be combined to obtain the greatest possible negative moment.

In combining the chord stresses due to bending with those due to the corresponding axial stress, care must be taken in determining the absolute maxima.

The stresses in the web members were computed by the method of moments, after the position of load giving the greatest stress had been determined by the aid of the locus-line.

The Deflections of the Trusses.—The deflections of the stiffening truss of a suspension bridge should not be considered as to the effects of the moving loads only. The condition of a suspension bridge balanced under the fixed load is that of an equilibrium polygon under stress. As stated in the foregoing, its stability is such that if disturbed by the advent of the moving load it will tend to get back into equilibrium, and this the more so the heavier the bridge. If, under the fixed load, the bridge is balanced with the horizontal pull in the cable, H_f, and the moving load afterward causes an additional pull, H_m, with a resulting deflection, δ_x, the bending moment produced by this condition in the stiffening truss will be:

$$M^1_x = \mathfrak{m} - H\,y - (H_f + H_m)\,\delta_x \ldots\ldots\ldots (34)$$

The first two expressions on the right-hand side of Equation 34 represent the moment as determined by the formulas deduced in the foregoing, while the third expression represents what we may term the "relief moment." The latter is caused by the disturbance in balance of the bridge by the deflection, which is equivalent to an increased versine. The effect of the fixed load on the stiffness of a suspension bridge is shown clearly in Equation 34. The heavier the bridge, the greater, of course, becomes the value of H_f, and the greater is the relief moment. The effect of this moment should be computed and the moments and stresses corrected correspondingly.

The resulting deflection, δ_x, of course, will be the deflection of the stiffening truss due the moment, M_x. Now, the deflection of any beam, by the common theory of flexure, may be written:

$$\frac{d^2\,\delta_x}{d\,x} = \frac{M_x}{E\,I} \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

Integrating the same twice, we get for the deflection

$$\delta_x = \frac{1}{E} \sum \Sigma \frac{M_x a^2}{I} \dots\dots\dots\dots\dots\dots\dots (36)$$

The greatest deflection of the main span, due to moving load, will occur with the main span fully loaded and the remainder of the bridge unloaded. Substituting for M_x^1, in Equation 36, the moment due to uniform load on a simple beam, we get, for the deflection at the center of the span,

$$\delta_{\frac{l}{2}} = \frac{M^1}{f} \times \frac{1}{E} \sum_0^{\frac{l}{2}} \sum_0^{\frac{l}{2}} \frac{a^2 y}{I} \dots\dots\dots\dots\dots\dots (37)$$

where M^1 is the moment resulting from Equation 34.

Equations 34 and 37 determine the values of both the moment and the corresponding deflection.

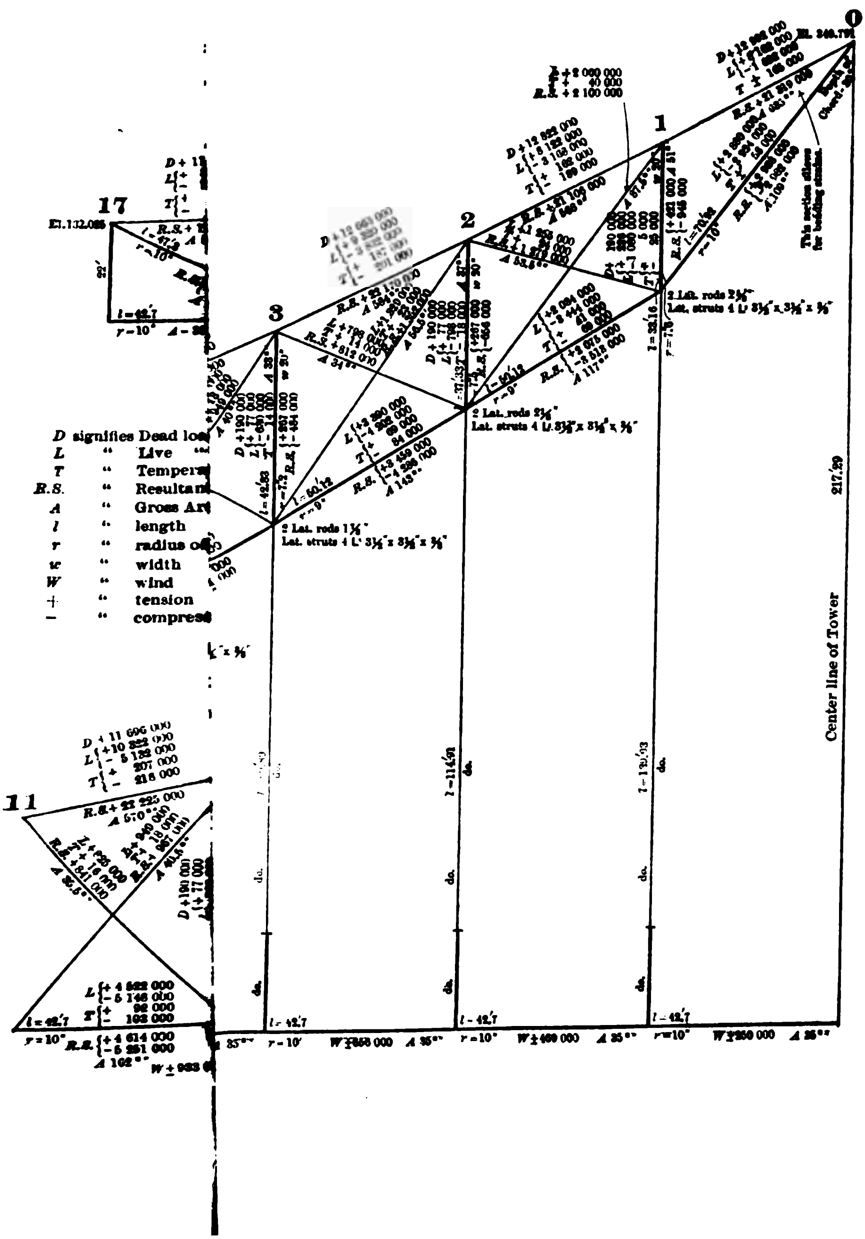
It will be noticed that no account was taken of the distortion of the polygon from its original curve. This has been done because, while for a cable not stiffened directly, the distortion of the original curve is of some importance, in the case described, its effect is negligible.

The Application of the Formulas Deduced to the Design of a Three-Span Suspension Bridge.

As stated, the formulas deduced in the foregoing were applied to a three-span suspension bridge of the form shown in Fig. 1. The bridge is a city bridge designed for a traffic capacity unparalleled in the annals of modern bridge building for similar purposes. It is intended to carry four trolley lines, four elevated railroad tracks, a wide roadway and promenades, and a working load of 8 000 lb. per lin. ft. of bridge, or a congested load of 16 000 lb. is provided for to take care of this traffic. A solid floor of buckle-plates is provided, and the total weight of the floor system, including pavement, rails, railings, etc., amounts to 16 300 lb. per lin. ft. of bridge. The cables and stiffening trusses were designed for the use of nickel-steel bars and built members. The total estimated fixed load on the bridge, including the floor, amounts to 31 000 lb. per lin. ft. on the main span and about 33 000 lb. on the side spans. The spans are 725, 1 470 and 725 ft., as shown on Fig. 1. A versine of 184 ft. was decided on for the main span, which, by the application of Equation 5, resulted in a side-span versine of 48.4 ft.

PLATE XXXVIII.
PAPERS, AM. SOC. C. E.
AUGUST, 1904.
MOISSEIFF ON
THREE-SPAN SUSPENSION BRIDGE.

trusses)
")



The proportions of the total horizontal pull, due to moving load or temperature, taken by the cable and stiffening chords are:

Main span: $m = 0.82$; $n = 0.18$;

Side spans: $m_1 = 0.89$; $n_1 = 0.11$.

It was found that, with a uniform moving load on the whole of the main span, about 8.5% of the load is carried by the stiffening truss acting as a beam, and the remaining 91.5% by the chains and chords acting as suspended polygons. The greater stiffness of the side spans, due to their short length and great depth at the center, shows its effects in the results obtained for the same kind of loading on them.

The side-span truss carries 98.6% of the load as a truss, and transfers 1.4% only to the cables.

The work on the computations presented no special difficulties. The summations which appear in the formulas deduced in this paper were computed and tabulated for every panel point, and the actual work was less formidable than the long equations would make it appear. The computation of the moments and stresses was simple work, and care only had to be exercised in obtaining the greatest resulting stresses. The analytical method displayed its advantages during the work of computation. The formulas being once established, the actual labor of tabulation can be carried on by several persons at the same time, each taking a part of the work, and, while it is desirable that each computer should be thoroughly acquainted with the formulas and their deductions, as was actually the case, such knowledge is not absolutely necessary. The work can thus be pushed ahead quite rapidly should the time allowed require it.

The stress sheets, Plates XXXV, XXXVI, XXXVII and XXXVIII may serve to show the relative amounts of the stresses and their variations, and also what may be expected under the given conditions.

Vol. XXX. AUGUST, 1904. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE COLLAPSE OF A BUILDING DURING CONSTRUCTION.

Discussion.*

By Messrs. Nathaniel Roberts, C. J. Tilden, F. T. Llewellyn, James P. Whiskeman, Guy B. Waite, C. C. Schneider, Oscar Lowinson, H. P. Macdonald, J. H. O'Brien and George A. Just.

Mr. Roberts. Nathaniel Roberts, M. Am. Soc. C. E. (by letter).—Mr. Parsons' paper should lead to some very interesting discussions on building construction in general and the use of cast-iron columns and details for high buildings, and be productive of good to engineers and the public at large.

When iron construction of buildings was in its infancy, and buildings, usually, did not exceed six stories in height and were very seldom of fire-proof construction, cast-iron columns were permissible. Their cost, compared with riveted columns, was greatly in their favor; also, the facility with which they were obtained and their ability to withstand fire without collapse, added to the large factor of safety in use twenty-five years ago (a factor of safety of 8 instead of 6, as at the present time), gave a safe construction.

In the early Seventies, the writer designed a large woolen mill in the West of England, the roof and floors of "slow-burning" construction, supported by cast-iron columns. After being in operation for several years, and the floors being saturated with oil, the mill

* This discussion (of the paper of H. de B. Parsons, M. Am. Soc. C. E., printed in *Proceedings* for April, 1904), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.
Communications on this subject received prior to September 23d, 1904, will be published subsequently.

caught fire in the upper story and burned downward like a torch. Mr. Roberts.
The 3-in. flooring was entirely destroyed, the 6 by 12-in. joists and the 10 by 16-in. oak girders were charred through, the several lines of heavy wrought-iron shafting were twisted and bent into various serpentine shapes, but the cast-iron columns, although subjected to a prolonged white heat and to streams of water from the fire engines, which caused most of them to crack from top to bottom, remained in place and prevented the collapse of the building, thereby saving the lives of scores of men who were engaged in removing goods from the lower floors.

The New York City building law requires that all wall columns, whether of steel or cast iron, shall be protected by

> "A casing of brickwork not less than 8 ins. in thickness on the outer surfaces, nor less than 4 ins. in thickness on the inner surfaces, and all bonded into the brickwork of the enclosure walls."

Also, all interior columns,

> "Used to support any fire-proof floor, shall be protected with not less than 2 ins. of fire-proof material, securely applied," but, "the extreme outer edge of lugs, brackets and similar supporting metal may project to within $\frac{3}{4}$ in. of the surface of the fireproofing."

It is a well-known fact, however, that many of the so-called fire-proofing coverings in use are fireproof in name only, as was shown by collapsed steel columns in the fires at Pittsburg and Baltimore.

The writer is not an advocate of the use of cast-iron columns in "tall" buildings, or even in buildings of moderate height, if subjected to bending strains caused by wind or eccentric loading, but believes that no one will dispute the fact that cast-iron columns of tough, gray iron, free from injurious cold-shuts or blow-holes, will withstand the combined action of fire and water without collapse, while steel columns will not.

The writer has no desire to criticise this paper, but wishes the author had gone into detail a little more, as it seems to be too general.

Mr. Parsons states that the building was constructed on the "cage" system, that is, all the weight was supported on the columns. Then he says, the "walls were continuous from the foundation, and did not rest on girders at the floor levels." In Fig. 3, however, twelve bays have wall girders for "skeleton construction," while the other outside bays are shown as being of "curtain-wall construction."

Mr. Parsons fails to give any data respecting the floor loads used in designing, and, although he gives a specimen column schedule in Table No. 1, he does not locate the position of the column in the building, to enable anyone to check the sections, and the detail of the 8-in. column, shown in Fig. 1, does not refer to one of the columns given in the specimen schedule, but appears to be either Nos. 29, 30, 35 or 36, at about the 4th story.

Mr. Roberts. He also states: "All the loads were eccentrically supported on the side brackets," while a reference to the plan, Fig. 3, shows that the interior columns and quite a few of the wall columns have very little eccentric loading.

He also states that the holes in the girders were about $\frac{7}{8}$ in. in diameter, and that the bolts were $\frac{3}{4}$ in. It is to be regretted that the exact size of the holes was not given. The standard size for punched holes for $\frac{3}{4}$-in. bolts is $\frac{13}{16}$ in. on the punch side and $\frac{7}{8}$ in. on the die side, although some contractors, to facilitate erection and cover careless workmanship, use a $\frac{7}{8}$-in. punch for $\frac{3}{4}$-in. bolts, *viz.*, $\frac{15}{16}$ in. on the punch side and 1 in. on the die side.

By reference to Fig. 3, it will be seen that Columns Nos. 31 and 37 are stayed in one direction only, the nearest beam in the other direction being at a distance of 1 ft. $9\frac{1}{2}$ ins. from the center of the column.

Fig. 3 is a typical plan of floor beams for the 3d to the 8th floors, inclusive. *Engineering News* of March 10th, 1904, gave a plan of the same floors, and stated that "the 9th to 12th floors and the pent-house floor are similar in arrangement to that shown, but their framing is somewhat lighter," and "the 1st and 2d floors, due to the addition of framing for the light-courts, are different in arrangement from the tiers above, and the 1st floor is heavier." The floor loads are also given as 60 lbs. live load and 60 lbs. dead load per square foot.

The floor construction is specified as the Roebling, flat, reinforced-concrete type (System B, Style 4), the fire-proof partitions being carried by the floor system.

The thickness of the floors is given as 13 ins. from finished ceiling to finished floor surface. This construction will weigh not less than 65 lbs. per square foot in addition to the weight of the fire-proof partitions; but the floor beams shown have been designed for a gross load of 120 lbs. per square foot of floor.

Engineering News also gives a schedule of sizes for Columns Nos. 30 and 31, and loads on Column No. 31, but it is to be regretted that enough data are not given to check some of the wall columns under eccentric loads, say Column No. 2, which is subjected to a bending moment at each tier due to an eccentric load of 11 400 lbs. To withstand this bending moment requires an increase in metal area in the upper columns equivalent to an added load of about 36 000 lbs. in addition to the actual load itself; while Column No. 1, with an eccentric load at each tier of only 5 600 lbs., requires an increase in metal area in the upper columns equivalent to an added load of about 32 000 lbs. in addition to the actual load itself.

In regard to the failure of the structure the writer believes there were several contributing circumstances, *viz.:*

1st.—The columns should have been designed for bending due to eccentric loading, necessitating steel columns, as no dependence can be placed upon a column to resist bending when it may be full of concealed blow-holes. Mr. Roberts.

2d.—The absence of bracing for wind strains, and the torsion on the structure caused by derricks, during the hoisting of material.

3d.—The overloading of the upper stories by building material may have had some effect; but, from information thus far obtainable the writer believes that torsional strains produced by derricks caused the collapse.

Only a few months ago the writer had his attention called to a building in course of erection in New York City, where steel columns in two-story lengths were used, the erector had the guy-lines for the derricks secured to the tops of the columns at least 10 ft. above any floor beams erected, and the erector was surprised (he ought not to have been) to find that some of the columns had a decided twist in them, and in some cases were more than 5 ins. out of plumb.

During the past fourteen years the writer has designed more than thirty steel buildings in New York City, ranging from twelve to thirty stories in height, besides many smaller buildings, and has found that, although there are some architects who realize the necessity of employing engineers to assist them in designing any but the simplest of steel constructions, the great majority of architects will with the aid of a rolling-mill handbook and the free services of a contractor's "iron-man," concoct a "design" that Providence sometimes allows to be erected without accident.

In conclusion: Mr. Parsons states that some of the beams and channels were marked "Phœnix" and some "Carnegie." Why were not some "Pencoyd" beams used, also? For is it not written, "a three-fold cord is not easily broken."

C. J. Tilden, Assoc. M. Am. Soc. C. E. (by letter).—The writer is strongly opposed to the use of cast iron as part of the framework in skeleton building construction. The principal objections are its general unreliability as compared with steel, and the apparent impossibility of designing satisfactory details for beam and girder connections, column joints, etc. The Hotel Darlington disaster, however, certainly does not prove anything against cast iron, *per se*, as a structural material; indeed, the columns seem to have acted remarkably well under the severe conditions imposed upon them from lack of lateral support and general bad design. Mr. Tilden.

The author makes note of the fact that each column length had top and bottom flanges cast only on the north and south sides, thereby giving the column no stiffness in an east and west direction, except that derived from the floor girders framing into the column near the top. The value of this stiffening may be seen by reference to Fig. 6. At *A* is shown one length of column, drawn to scale, and at *B* is an

Mr. Tilden. enlarged detail of the lug to which the floor girder was bolted. (This sketch applies to the columns numbered 7 or 15 in Fig. 3, showing a typical plan of the floor framing.) The bolts used were $\frac{3}{4}$ in. in diameter, and the holes in the lug and girder web were $\frac{7}{8}$ in. Assuming, when the column is vertical, that the bolts are concentric with the bolt holes, as shown by the heavy lines of Sketch *B*, Fig. 6, a horizontal thrust against the end of the girder would tilt the whole column east or west until the axis of the bolt holes, shown at $x\ x$ in normal position, assumed the position, $x'x'$, the bolts in this position bearing against the bolt holes on opposite sides, as at *M M*. A movement of this kind and amount would be resisted only by the friction due to tightly set-up bolts, and it is admitted that in most cases these were loose. The effect of such motion on the column is shown in Sketch *A*, Fig. 6. In one story length the column could readily be moved 3 ins. out of plumb. Such a condition of affairs could hardly have existed if steel instead of cast-iron columns had been used in the framing.

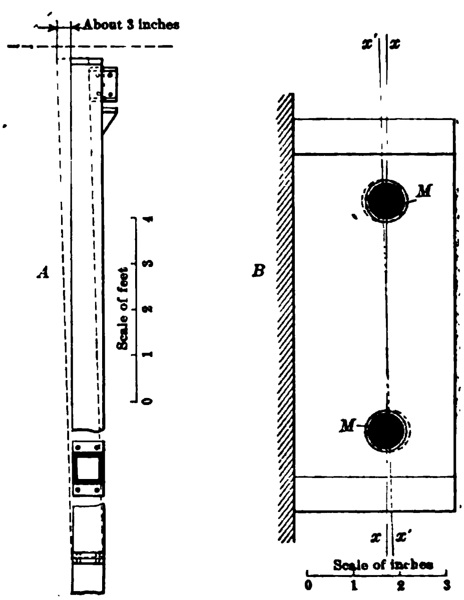


Fig. 6.

Referring again to Fig. 3, it is seen that the foregoing reasoning would apply to the two rows of columns numbered 7, 31, 30, 29, 28; and 15, 37, 36, 35 and 22. That is, practically the whole framework could be tilted east or west until it was out of plumb by an amount equal to 3 ins. for each story length. As a matter of fact, it was brought out in the testimony before the coroner's jury that, a week or two before the collapse, the structure (then about seven or eight stories high) was out of plumb by an amount variously estimated at from 6 ins. to 2 ft. This error was corrected by rope tackle used to jack the leaning framework back into place. The direction of this inclination is not mentioned in the press reports, so that it cannot be stated definitely whether or not the trouble was due directly to the cause just cited. The diagram shows rather strikingly, however, the utter flimsiness of the whole structure, and it is easy to credit the further testimony of witnesses before the coroner that on various occasions the building swayed noticeably.* It is perhaps hardly necessary to speak of the severe shearing stress to which the bolts would be subjected by any distortion of the nature and amount indicated. Mr. Tilden.

One direct violation of the building law may be noted. The statute† reads:

"The top and bottom flanges, seats and lugs (of cast-iron columns) shall be of ample strength, reinforced by fillets and brackets; they shall be not less than 1 in. in thickness when finished."

The flanges of columns in the Darlington were not reinforced in any way, and the fact that nearly every column broke in the flanges indicates how serious a defect this was. At the inquest the foreman of the iron workers testified that the Building Department had ordered him on several different occasions to stop work on the iron frame, and that in each case the owner had told him to go ahead regardless of these orders.‡

The real trouble, therefore, would seem to be the lack of a sufficiently severe penalty for violation of the building code, and the subject would seem to be one for discussion by law makers rather than engineers. In relation to this, it may not be amiss to mention the recent discovery by an archæologist of the building laws which were in force during the reign of Hammurabi, King of Babylon, about 2250 B. C. The records§ of this wise and just monarch have been deciphered, and, among other ordinances, the one numbered CCXXIX reads:

"If a builder build a house and has not made his work strong, and the house has fallen in and killed the owner of the house, then that builder shall be put to death."

Further ordinances provide that the builder's son shall suffer death, if through his father's negligence the son of the householder is killed, and so on, in strict conformity with the Mosaic doctrine of "an eye for an eye," through the various members of the household. This bit of legal history is respectfully submitted to the law makers of Greater New York as worthy of the careful and reverent consideration to which more than four thousand years of "precedent" would seem to entitle it.

F. T. Llewellyn, Assoc. M. Am. Soc. C. E.—Upon reading Mr. Parsons' description of the collapse of the Darlington Building, and recalling the other detailed articles which have appeared in the technical journals, it is evident that this disaster was caused by the inefficiency of the columns, as stated in the author's concluding paragraph; Mr. Llewellyn.

* *Engineering News*, March 24th, 1904, p. 281.
† Building Code, Part XXII, Section 112.
‡ *Id. loc. cit.*
§ Editorial, *American Architect and Building News*, March 26th, 1904.

Mr. Llewellyn. and it is of interest to inquire if the Darlington was an isolated case of this kind. The writer has looked up all the kindred instances of similar disasters of which any record was available, and gives a short summary thereof, with his sources of information, so as to see what common factor, if any, there may be.

The earliest reference found is in an address* given by C. T. Purdy, M. Am. Soc. C. E., before the Boston Society of Civil Engineers, wherein he says:

"The days in which cast-iron columns will be used in the construction of high buildings are fast being numbered. It would hardly seem necessary to compare them with steel columns before an audience of New England men, if we will but recall the Pemberton mill disaster with its frightful loss of life."

The writer has not been able to secure any data regarding this disaster, and would be glad to learn the details.

In *Engineering News* of May 24th, 1894, is described the distortion by wind of the nearly constructed building at No. 14 Maiden Lane, New York City, due to the loosely bolted connections of the cast-iron columns, although in this case they were much better than usual, and the framework was subsequently pulled back.

In *Engineering News* of December 6th, 1894, is given a short account of the failure of the five-story building, belonging to the Montreal Street Railway, which also had cast columns, and which, apparently, gave way after completion by the crushing of a wall.

Regarding the collapse of the nearly finished Ireland Building on West Broadway and Third Street, New York City, *Engineering News* of August 15th, 1895, says: "The prime cause of the accident appears to us to have been the weakness of the cast-iron columns." In a later issue, August 29th, additional evidence seemed to show that faulty foundations were the primary cause.

Commenting on the failure of the Brown Soap Factory building, on Twelfth Avenue, between Fifty-first and Fifty-second Streets, New York City, *The Engineering Record* of June 12th, 1897, after criticising in detail the cast-column construction, says: "Cast-iron columns are a source of great danger wherever they are used, as we have frequently shown, and no safety factor can remove that danger." And *Engineering News* of June 17th, 1897, concludes an exhaustive review of the same structure with the following:

"Since the failure of the Ashtabula Bridge, twenty years ago, no bridge engineer has dared to risk the safety of a bridge upon the lug of a cast-iron column. Will it require a disaster of similar extent to awaken engineers and architects to their fatuity in continuing the use of such cast-iron columns with all their hidden defects and their eccentrically loaded lugs in buildings the failure of which may cause the loss of hundreds of lives?"

The collapse of the Darlington Building, in which the columns were cast iron, is added to complete the list.

* *Journal of the Association of Engineering Societies*, March, 1895.

Mr. Llewellyn.

It is noted, of course, in each of the cases cited that cast-iron columns were used in every instance. After a search through all the accessible records, the writer has been unable to find any case of failure through the use of steel columns, and if such exist he would appreciate a description of the details. Although some of these failures were probably induced by other causes, many of them seem to have resulted directly from defective cast columns, and in all cases the collapse was intensified by their lack of stiffness. It may be of interest, therefore, to summarize briefly the properties of cast iron as used in the columns of high buildings, passing over the uncertainty regarding hidden internal defects in their pouring and cooling as sufficiently well known, and assuming their manufacture and details to be up to the standard.

1.—It is impracticable to connect a cast-iron column with the ones immediately above or below it by any kind of splice joint which will preserve their continuity, as, on account of the danger of cracking the castings by riveting, the flanges must be bolted, and, almost invariably, the bolts are a loose fit. Also the weakness under tension of the cast flanges (which it is very hard to pour sound) makes them a poor medium to transmit the pull of the bolts. Hence a stack of cast columns is little more than a lot of those toy wooden pillars we used to pile end on end when we were children.

2.—This instability could be decreased if each story column were braced by a rigid connection with the horizontal girders and beams, but, again, the impracticability of riveting and the tensile weakness of the cast lugs result in a loose joint, which leaves the effective unbraced length of the columns the full height of the building instead of story lengths, and throws out the usual formulas of strength.

3.—The best specifications require the projecting brackets to be given a slight slope downward, in order to throw the load of the supported beams as close as possible to the column and relieve the tension on the cast shelves, but, in ordinary foundry practice, the accuracy of this slope cannot be ensured. Neither does it seem desirable to reinforce the column at the brackets by an internal diaphragm, for, apart from the additional expense in the moulding, there would arise such uneven cooling stresses as would weaken, rather than strengthen, that place.

In a word, cast iron has proved itself to be a dangerous material for use in the columns of high buildings, and the Darlington disaster, as reported by Mr. Parsons, is merely one more object lesson. Can we afford to look with unconcern upon such construction for those members upon which depend the entire strength of our buildings and the safety of much life and property? All railroad engineers have long since vetoed the use of cast iron for bridges, and, neglecting the commercial side, united effort should be made to bar this material

Mr. Llewellyn. from use as columns in any but low and simple buildings. It is understood that, among other things, the prudence exhibited in the design of railroad bridges is recognized by the insurance companies, who place a premium upon the security of railroad travel by paying twice the amount of their ordinary policies in case of accidents while traveling on the railways. The same protection, of course, would be best afforded by the employment of only competent engineers to co-operate with architects in the design of tall or hazardous buildings, but, as this desideratum cannot always be controlled, the building codes of our great cities should be amended so as to make impossible the flimsy structures now only too common. As stated in a recent editorial on this subject, with steel columns a wholly satisfactory design is possible in all cases, while the limitations of cast iron are firmly fixed.

No reference has been made to the action of cast iron in fires, nor to its durability against corrosion, nor to its actual strength, these matters being foreign to the paper, but it is suggested that the whole question of the behavior of cast iron, steel and other columns, under the various possible conditions, would form a most interesting and pregnant subject for another discussion. The writer would commend this to the notice of the proper committee.

Mr. Whiskeman. James P. Whiskeman, Assoc. M. Am. Soc. C. E.—The speaker was detailed to make a report on the collapse of the so-called Darlington Hotel, and was on the premises until the wreckage was all cleared away. Without going into details, which are covered by Mr. Parsons' paper, the following can be added to what has already been stated.

The details for the framing were defective in the following respects: Not all the columns were eccentrically loaded, but the majority of them were. The worst cases were in the wall columns. The seats for the channels nearest the walls (which were also girders) projected some 9½ ins. from the face of the column. To this column the channels were bolted with one ¾-in. bolt about 8 ins. from the face of the column, and with no bracket connection. The plans showed these channels framing into a 9-in. I-beam, which was to be framed close to the column, on proper seats and with a standard bracket. In curtain-wall construction, eccentric loading is quite common, as it is desirable to have the columns embedded in the walls without offsets or projections on the interior of the walls. The columns supporting the walls were eccentrically loaded, but the girders remained level and the columns plumb after the fall.

The column flanges should have extended around the four sides of the columns, instead of on two sides only. This, together with proper bolting, would have given additional strength and rigidity to the framing.

The interior columns on the front and rear were not braced properly in a longitudinal direction. The beams, instead of framing into the columns, were framed to the girders on each side of the columns, and, therefore, were not properly supported in this direction at every floor. The majority of the columns broke at the flanges. The bolting appeared to be loose, and, as the threads were not stripped, it would appear that the columns were not properly bolted together. The bolts on one side were probably loosened in plumbing, and, as no shims were used, the load was concentrated more or less on the opposite flange. The metal in the flanges was honeycombed and full of impurities. Mr. Whiskeman.

There was also loose and insufficient bolting in the beam and girder framing. In one case three bolts were omitted from a possible four; in another case two bolts out of a possible four. But even where these bolts were omitted the parts remained framed together. An 8-in. I-beam was found in the wreckage bolted at one end to part of a 7 by 7 by 1-in. column with one bolt, and to a lug of a column at the other end with one bolt. This beam had two holes at each end close to the bolt holes and was weak, but the connection was stronger than the columns.

The cinder concrete of the floor arches was disintegrated in the fall, but this was due to the fact that it had not set properly. The centoring had not been removed. Several specimens of this concrete preserved in a warm room became very hard in a few days.

As to the amount of material stored on the floors of the building, owing to frequent complaints from the neighbors, material was removed from the street as soon as possible after its delivery, and stored in the building. On the first floor, back of the light-shaft and against the west wall was piled a lot of Rosendale cement in bags and barrels, producing a live load of 250 lbs. per square foot on this floor, which was calculated to stand safely 60 lbs.; but the floor did not fail on this account. On the upper floor was distributed, ready for erection, or in piles, considerable iron and steel. In the neighborhood of Column 37, about twenty pieces of unerected iron were found in the wreckage. About 30 tons of unerected material were found in the ruins.

The derrick used for hoisting iron was supported on the top of the framework, and in the center of the building near the front. The derrick had a boom probably 60 ft. long. How well the weight of the derrick was distributed, or the framework temporarily reinforced, it is hard to say, but it can be surmised that it was not properly done. It was also probably guyed to the structure in such a way as to strain it excessively. The derrick was not in use on the day of the collapse. There were frequent complaints from the adjoining tenants that the hoisting of materials in the elevators caused their buildings to vibrate.

The building was out of plumb at various times from 6 to 18 ins., Mr. Whiskeman. and was never properly sway-braced before or after it was pulled back. Excessive flange strains were produced in this way, which could cause the building to collapse at a later date. Columns 36 and 37 were not braced properly longitudinally, and should be treated as long columns, and one of these broke about half-way up, or in the 5th story, and fell. On account of the lack of rigidity, the entire mass was precipitated toward these columns. Columns 34, 36 and 37 were the only ones broken off in the basement. None of the foundations failed in the slightest degree.

The overloading of the floors, by the iron or by the derrick, the loose bolting of all members, the lack of longitudinal supports for the columns, the building being out of plumb and never properly sway-braced, all combined to cause the complete collapse, which any one of them was sufficient to induce.

The structural engineer has learned nothing new from this collapse, as some or all of the defects noted can be found in other buildings of the same class.

The designing of buildings of this class is considered in much too light a vein, and, as the operation is more or less a speculation, the temptation exists to make the structure too light at the expense of rigidity and stiffness. On account of the height being less than four times the width, no permanent wind bracing is provided and as little temporary bracing as can conveniently be put in. Reliance is placed on the curtain walls and fire-proof floors to stiffen the structure, but, in reality, the framework is sometimes nearly completed before the walls are begun, therefore elements of strength are depended upon which rarely exist until the building is completed.

Cast-iron columns are not readily adapted for the design of any other than direct central loading. It is difficult to provide good details, and, in some cases, impossible when the loading is eccentric and there is little rigidity against forces tending to distort the structure. The factor of safety generally used is about 2.7, which is not enough for any columns except those centrally loaded. It would be good practice to limit their use to buildings of 75 ft. in height, and to buildings of greater height only when they have a very broad base.

Section 112 of the New York Building Code prescribes the minimum size of cast-iron columns as 5 ins. and the minimum thickness as ¾ in. In the upper stories of buildings of this class, on account of the light loading, it is found cheaper to use light steel sections in preference to cast iron, and single I-beams, channels and two angles, therefore, are often used for columns. Little attention is paid to detailing, excepting to put in connections a sufficient number of rivets for shear and bearing. These columns are little or no better than if cast iron were used, and are sometimes so slender as to make them less desirable. The columns, taken as a whole, from top to bottom are plumb, but zigzag from story to story.

In a building which came under the speaker's observation recently, the equivalent direct load due to an eccentricity of about 16 ins. was more than the direct load on the column. Mr. Whiskeman.

The following illustration will show to what extent the saving of material is carried In a building of this character, just now being completed, the girders next to the walls, which were very eccentrically framed to the columns, were, in addition, cantilevered on each side beyond the column, so that the spans of the adjacent girders were reduced by the amount of the projection, and lighter sections were used in these panels.

In another instance, the cast-iron columns were eccentrically loaded simply because the girders had been ordered too short and the column seats and brackets were extended to meet the short girders.

It is, perhaps, a little foreign to the subject, but, in tenement-house construction, even more so than in the apartment hotels, the proper detailing of iron is not more than approximated. In a corner tenement house the entire first story on two sides is left open for stores, and the walls above are supported on girders and columns, the columns resting on piers in the basement. The columns are rarely provided with shoes or caps; the girders simply rest on the columns, and are not bolted to them. Where two sets of girders meet, they are strapped together with a narrow plate placed across the joint diagonally and bolted somewhere on each side with one bolt, usually the separator-bolt. This joint could open 1 in. without even straining the strap. No attempt is made to tie or anchor the columns, and, when a pier settles, on account of the lack of rigidity, the building partly or wholly collapses.

In one case, a narrow column, supporting a front and not provided with a bottom flange or shoe, on account of the concentrated load in the center of a poorly-bedded pier cap, cracked the cap and the front collapsed.

In another instance, practically the whole of a six-story tenement house, in process of completion, collapsed by the undermining of a single pier.

There is practically no engineering at all in tenement-house construction.

The design for the framing of apartment-hotel buildings is a problem for treatment by a competent structural engineer, but is more often solved by the use of tables which are but poorly understood and applied accordingly. In some cases the engineer or architect who knows better will prostitute good design to cheapen the cost.

This is one-half the solution of the problem; the solution of the other half is the employment of a competent superintendent on the

Mr. Whiskeman. premises to see that the work is executed properly and that no undue strains come upon the uncompleted structure.

The solution of this problem is competent engineering on the designs and proper superintendence in the field.

Mr. Waite. GUY B. WAITE, Assoc. M. Am. Soc. C. E. (by letter).—The general conditions of construction at the Hotel Darlington appear to be about the same as the average conditions in the construction of most of the apartment hotels erected in New York City at the present time.

Since the building laws of New York City have permitted non-bearing walls to be carried on their own foundations, most of these hotels have been constructed with steelwork to carry the foundations, only.

The steel-floor systems are designed to carry live loads of 60 lbs. per square foot, and dead loads of from 60 to 70 lbs. per square foot, making a total of from 120 to 130 lbs. per square foot.

To carry these floor loads economically, the floor beams are generally spaced at about 6-ft. centers.

The interior columns are studiously located by the architect, who places them as much out of the way as possible, in partitions, etc. The outside columns can usually be located with greater freedom than the interior columns, and may be located to get better results for stiffening the building laterally.

A paper* bearing on the subject of constructing high buildings for resisting lateral forces was presented to this Society in 1894. This paper was presented at a time when the construction of high buildings was comparatively new, and there was no specific building code in New York City regulating them. There were no established precedents among designers for the lateral stiffening of the buildings, and there was no law referring thereto.

The writer, who then had the examination into these constructions for the Department of Buildings, was obliged to act arbitrarily on eccentricities of construction. There were many narrow and high buildings proposed, without, as he believed, safe construction for resisting lateral forces. The discussion which followed this paper applies with equal force to the Hotel Darlington and similar buildings.

In the paper referred to, all parts of the finished structure which were capable of aiding in lateral resistance were considered. Partitions, walls, and steelwork, with the connections, were the principal factors. Partitions are almost insignificant factors in ordinary building construction. The walls and the steelwork are the important factors for lateral resistance, as usually constructed. If the walls are not pierced too much with openings, they, undoubtedly, form the main factors in actual practice.

Where walls are to supply the principal lateral stiffness to a build-

* *Transactions*, Am. Soc. C. E., Vol. XXXIII, p. 190.

ing, it would seem as though these walls should be carried up with the steel construction, or that suitable temporary lateral braces should be provided until the walls can be carried up to the top of the steel construction. Mr. Waite.

While walls appear to be generally relied on to furnish the lateral stiffness of buildings, it is believed that the Hotel Darlington disaster shows the danger of depending too much on this one factor.

The writer believes that the steel construction with its connections should be sufficient to hold the steel skeleton plumb until the walls can be carried up.

As deduced in the former paper, lateral forces may sometimes develop stresses (which increase from the top toward the bottom of the building) which become excessive if the building is very high.

In the Hotel Darlington, as the conditions are understood, the walls not being carried up with the steelwork, the columns and girders with their connections formed the main resistance to lateral forces from the top down to the place where the exterior walls were finished. These lateral forces were probably sufficient at or about the 4th floor (where the wall was stopped) to break the cast-iron columns at their connections.

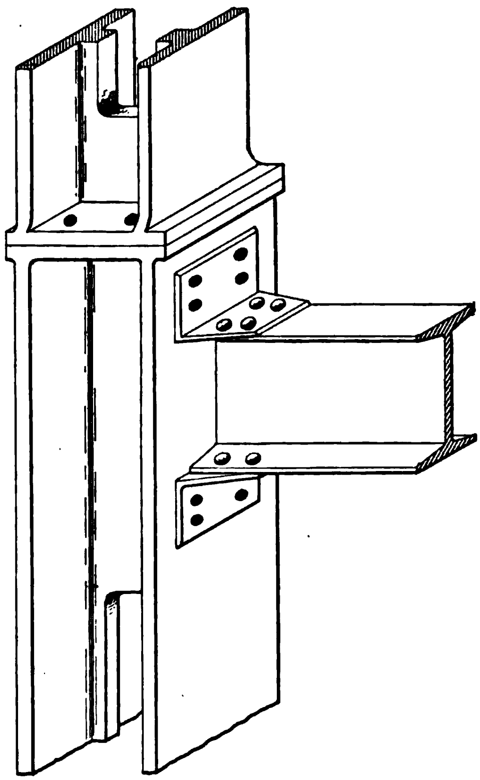


FIG. 7.

The two re-entrant light-court walls, located midway between the front and the rear of the building, were favorable features of construction, adapted to give lateral strength to the narrow way of the building.

Had the structure been held in position until these walls were constructed, and until the floor arches were solid and capable of acting in the capacity of horizontal trusses—conveying lateral stresses to

Mr. Waite. points of greatest resistance—probably nothing extraordinary would have been heard of the Hotel Darlington.

While the spacing of floor beams and the location of interior columns was bad, in that there was not a direct connection with the columns in every case, this need not be a defect in the construction. Providing other parts of the structure have requisite lateral strength, and the floor system is capable of transmitting the lateral forces to these stiffened parts, these interior columns need only be supporting columns.

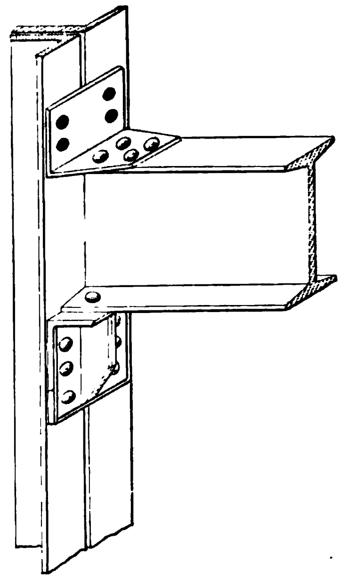


FIG. 8.

The writer now believes, as firmly as he did when preparing the paper ten years ago, that not providing for lateral resistance in steel construction is dangerous.

The writer was associated for several years with iron and steel contractors who were also engaged in the manufacture of cast iron, and therefore, they desired to use cast-iron columns where practicable. The practice was to use cast-iron columns for heavier loads, in lower stories, and steel columns for the lighter loads, in upper stories—the latter being more economical for very light loads than the former.

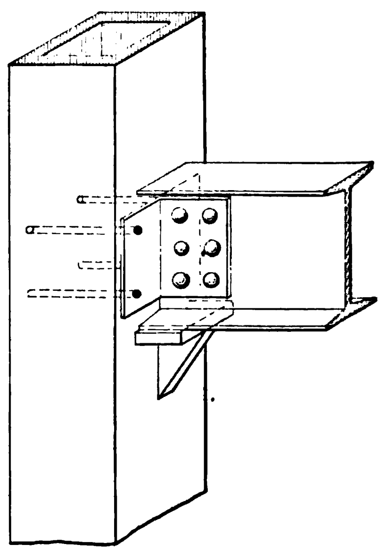


FIG. 9.

In order to insure positively rigid connections, for resisting lateral forces, steel connections were riveted to the beams (which were to be connected to columns) in the shop, and were connected to the cast columns, as shown in Figs. 7, 8 and 9, by bolts which could be brought up taut.

The connections shown in Fig. 7

were used in the narrow skeleton- constructed buildings No. 64 Fulton Street. 25 ft. wide and nine stories high, and No. 708 Broadway, 25 ft. wide and ten stories high, where beams were spaced at about 6.5 ft. centers and were connected at each end to cast-iron I-shaped columns in the lower stories, and to channel-steel columns in the upper stories. Where the floor loads were too heavy for the bolts shown in Fig. 7, connections were made to the column seats as shown in Fig. 8. Mr. Waite.

Rigid connections have been made to cast-iron columns by shop-riveting angles to the sides of girders, and bolting through the columns, as shown in Fig. 9.

The flange punching for the connections shown in Figs. 7 and 8 is an additional expense, but, where any great resistance is required, the advantages gained will more than compensate for this cost.

C. C. Schneider, M. Am. Soc. C. E.—This paper illustrates only one of the many examples of poor building construction, and also one of the many abuses of cast iron in such structures. Mr. Schneider.

A careful observer will notice many structures, similar to the Darlington Hotel, which would have collapsed under similar conditions.

In examining the design, the speaker was impressed with the incompetence of the designer, his lack of knowledge of even the first principles of structural designing and his presumption in undertaking a piece of work for which he was not fitted.

The speaker is gratified to know that no member of our profession had anything to do with the designing and construction of this structural monstrosity, but that this design was made by a mechanic whose experience and knowledge of building construction was very limited.

In designing this structure, no attempt was made to insure lateral stability or provision for wind pressure during erection.

The speaker endorses the author's conclusion, that the structure collapsed because of lack of lateral support for the story columns.

The metal framework of a building of the skeleton type of construction is a complete structure in itself during erection, and should be treated as such; that is, it should be self-supporting before the walls and floors are completed, and in that condition be strong enough to resist the wind forces on all exposed surfaces, and such additional strain as may be expected during erection.

The speaker has always adhered to these principles in his own practice, and has found that if provision is made for the usually assumed wind pressure of 30 lbs. on every square foot of exposed surface of the metal framework, the structure is generally strong enough to resist such additional strain as may occur during erection; for instance, the lateral strains produced by a swinging derrick, etc.

There is one feature in the metal structure of the Darlington Hotel which deserves severe criticism, and that is the use of cast-iron columns in a building about 144 ft. high.

Mr. Schneider. Experience has taught us that cast iron is unsafe and unreliable as a material for structural purposes, and, for that reason, it has been entirely discarded in the construction of bridges of all kinds for many years.

In this case, cast iron was used in combination with poor details.

The designer followed the foundryman's practice of carrying the beams or girders on brackets cast on the columns, which, however, the speaker does not consider good practice.

The brackets are generally unreliable, as very frequently they are full of blow-holes, without showing any defect on the outside; this fact is also verified by the author's observation.

In view of these ever-occurring disasters, it seems that something should be done to protect human life against accidents of this kind, which might have been avoided if such structures were properly designed and inspected.

A competent and reputable engineer needs neither building law nor supervision; however, as we cannot compel owners of buildings to employ competent engineers, it becomes necessary to have proper and strict supervision over all structures of this kind.

The people generally rely on the building departments for protection against unscrupulous contractors and ignorant pretenders. Therefore, the building department of every municipality should employ a sufficient number of competent engineers to enable them to examine the designs properly and carefully and inspect the work of all structures within its jurisdiction.

This supervision should not be perfunctory, but thorough. If a design is not safe, it should be rejected, even if it conforms to the letter of the building law.

It should be the duty of the building department, and that department should have the power to see that all structures are safe, first, last and all the time, with or without building laws.

The indiscriminate use of cast iron should be prohibited. It should be confined to ornamental work, column bases, or such other parts as are only strained in direct compression.

Cast-iron columns should be ruled out in all buildings over four stories high.

Mr. Lowinson. Oscar Lowinson, Assoc. M. Am. Soc. C. E.—The discussion thus far has demonstrated that certain fundamental errors in construction were committed in the erection of this building. It has also brought out the fact that engineers, apparently, are not agreed as to what constitutes safety in construction.

One sees public departments of a great city, under whose absolute control the construction of buildings is supposed to be, permitting to be erected structures which engineers have stated should not have been built.

The fact that the Darlington was constructed with cast-iron columns has brought out the statement that such a building cannot be built and assurance be given that it will be safe. Mr. Lowinson.

Some thirty years ago, because of a disaster caused by the collapse of a bridge, our bridge engineers decided almost unanimously that cast iron has no place among the main structural members of a bridge subject to shock; so that, when we hear prominent engineers state that cast iron, by reason of its unreliability, has no place in a building, there is room for serious reflection and discussion.

The problems connected with apartment hotel construction in New York City are somewhat different from those in commercial buildings. In the first place, the buildings being purely speculative, the designers have been compelled to exercise every device possible to enable them to secure the completion of the buildings at a minimum of cost; and, as the buildings must be sold as soon as possible, there are no responsibilities other than moral, and as long as the building stands together until sold, the only controlling influences are the conscience of the owner and the restriction of the public authorities.

The result of this striving for economy is seen in the forms of construction adopted by structural designers. In buildings of this class, of which there are a great many either lately finished or now in process of construction, cast-iron columns are generally used in the lower stories, and, because of the restrictions of the building code as to the minimum diameter and thickness of columns, the five or six upper stories of these buildings are usually supported on structural members, which are either I-beams, channels, or, in some cases, even single angles, so that, on seeing such a structure, one cannot comprehend how it is physically possible to transfer the load to these columns, even though they are able to carry it if it were once properly placed upon them.

A statement of the conditions existing in connection with the construction of buildings of this class, while not pertinent to this question, is deserving of recognition because of its effect upon professional work, and these conditions will be described briefly.

The Darlington is typical, and the owners of the building organized into a company for the express purpose of constructing it. In no way had anyone in this company ever been connected with the responsibility of the construction of a building before.

A firm of architects (whose sole business is to draw plans and make applications to the public departments for the necessary permits for construction) was employed, at a fee which was about one-tenth the schedule rate, adopted by the American Institute of Architects, for this service.

No arrangements whatever were made for superintendence, the owner doing his own superintending.

Mr. Lowinson. The owner takes estimate of both labor and materials on each item of the work. In cases where materials will be delivered to a contractor by a firm, the owner tries to get the contractor to supply these (subject to the lowest prices), as he is thereby enabled to shift the responsibility to the contractor. Otherwise, the owner supplies the material and the contractor the labor.

The owner depends on the municipal authorities to see that the labor will be performed properly and usually does not concern himself with them until he receives a notice from one of the public departments that the law is being violated and that he has incurred a financial penalty thereby. His interests are in the finished details, such matters as appeal to the eye and will help make a sale of the building.

It must be borne in mind that the owner's financial interests are usually very little, if any, and he looks to his profits by the increased amount received at a sale over the cost of the structure and land.

In the case in point, it is not worth while to enter into a detailed discussion of any other item than of structural ironwork.

The architects prepared a set of framing plans, and the Bureau of Buildings issued permits, on the assumption that the details did not require examination.

Because of the intimate relationship between the structural ironwork and fire-proof flooring systems, it has become customary for the firms installing the floor systems to prepare a framing plan most economical for their system. In this case, the various fire-proof firms easily showed the owners that they could save a sum (in the case of the system adopted) of $6 000 by using different spacing and lighter members in the floor construction.

Of course, there being no engineer or other intelligent person knowing anything about construction, this economy was adopted. (It appears strange that the owners of all these buildings never called their architects to task for this apparent waste of material and money.)

The iron contract for finishing and erecting the building was left to a firm composed of two persons, one of whom knew nothing about iron or buildings, other than would be known by any ordinary layman, and the other had had experience (as he stated at the coroner's inquest) in heating rivets at a forge, and helping screw up bolts on some large buildings where he had been employed previous to going into business for himself. Previous to taking this contract, they had built some fire-escapes and had taken a few small contracts. This was their first real venture.

They realized immediately that it was necessary to have help from somebody who knew something about ironwork, and a man was employed, at a salary of about $15 a week, for this purpose. This man, apparently, had had some experience, and, examining the plans

which had been prepared by one of the fire-proof companies, and Mr. Lowinson.
under which the contract was taken, protested that the floor members were too weak, and he prepared a new set of framing plans in which there was additional iron worth about $2 000. The total value of the contract was in the neighborhood of $28 000.

The cast-iron columns were left as marked on the plans originally. It does not appear that any investigation was undertaken to determine whether some more thousands of dollars might have been saved by scamping these.

The foregoing statement is typical, and there are serious doubts whether there is one building in ten of this class of construction erected under different auspices.

The beams were bolted to girders, girders were bolted to columns, and columns were bolted together, but in no part of the structure was any attempt made to secure the rigidity which is absolutely essential for a structure designed with the formulas used for that purpose; nor was any arrangement made to give assurance that the building must stand under any conditions to which it was likely to be subjected. The connections and the eccentricities of the load were not provided for properly. The flanges of the columns extended on two sides only, so that no safety could be obtained by their resting on secure bases, in case of slight motion, because of possible looseness in connections.

The workmanship in the erection was poor.

Some three weeks before the collapse, it has been stated by several eye-witnesses, the building was very much out of plumb.

The foreman of the erection gang stated that he believed it was 18 ins. out of plumb.

Apparently, the idea of danger never suggested itself to anyone, even after the leaning was known. To bring it back into plumb, cables and tackles were used. The bolts on the columns of the lower stories were loosened up, and the building was jacked back. There is no record that the columns were screwed up again, and the absence of shims demonstrated that no arrangements had been made for keeping the building plumb, when it had been brought back. Ordinary common sense should have dictated that somebody should be called in to find out the trouble and the remedy, but engineers cost money, and the speculative builder has no money to spend for such foolishness. He has been taking chances all his life, and this is a mere incident. The chances are that luck will not go against him this time.

The side walls of the building were up about three or four stories, the point at which the columns apparently broke off around the walls. The brickwork was laid in cement mortar, and appeared to be fairly well constructed.

Mr. Lowinson. The building was what was called a cage construction, which has another grave defect in that the wall columns are embedded in the walls, and, with the projections at their flanges and lugs, permit the wall in shrinking to throw upon the columns and footing considerable weight for which they were not designed.

There is no evidence, however, to show that fracture occurred in any of the wall columns, because of this concentration of the load due to the shrinkage of the walls and the non-shrinkage of the projections on the columns.

The main source of weakness, as stated by several engineers, was the fact that a cast-iron, unbraced building was erected nearly twelve stories in height with practically no provision to resist external strains due to wind and vibration. It had less strength, relatively, than a toy house, twelve blocks high, built with children's playing blocks. Because it had nothing to hold it up, it fell. The same may happen at any moment to any of the other structures of this class.

The speaker believes that the author of the paper states the primal cause of the collapse to be due to the failure of one of the third- or fourth-story columns. It is possible that that may be true. The author evolves a theory of the "Center of Fall," with which the speaker must confess that, because of his unimaginative mind, he cannot bring himself to agree.

Our laws are fairly good. It is a very difficult matter to frame laws so as to cover every detail of construction. This building has made manifest a number of weaknesses in the New York Building Code. The great lesson that it is hoped has been learned by this building collapse is the crime of constructing buildings with no intelligent supervision. The idea that a public department should assure the safety of the construction is also wrong. The Bureau of Buildings has certain functions to perform. Its duties are purely those of police surveillance. In Germany, for instance, the Building Inspection Department in each city is a branch of the police and is called "Bau-polizei," Building Police. The officers of this bureau should not be expected to guarantee the absolute safety of the structure, but they should be expected to detect flagrant violations. The Iron Inspector stated that he had about fifty buildings of this class in his district, and spent about ten or fifteen minutes per week at each building.

The main weakness in the law is that there are no means of compelling the class of people who are building now to employ competent men to undertake this construction.

It is an unfortunate fact that many buildings in this city are in danger of collapse at any moment.

Mr. Macdonald. H. P. MACDONALD, JUN. AM. SOC. C. E.—The speaker has been paying especial attention to the ratio of the thickness of metal in

Mr. Macdonald.

cast-iron column flanges and beam seats to that in the shaft of the column, as affecting the solidity of such members. When the metal in such a part is heavier than that of the shaft, it remains in a liquid state after the metal in the shaft is solid, and as it cools and shrinks is likely to draw away, near its center, from the shaft, causing a shrink-hole; such conditions also tend to the formation of blow-holes. The sample which Mr. Parsons has is a good illustration of this case, the metal in the flange before it was machined being probably 1¾ ins. thick, while that of the shaft is 1 in. Far more exaggerated cases of this error have come to the speaker's notice, some designers putting beam seats 1¾ ins. thick on shafts ¾ in. thick. The metal in a flange or bracket should never be more than one-fourth thicker than that of the shaft where it is located; and, where special strength is required, the shaft can be thickened, or two or more vertical ribs be used instead of one, as is the common practice. The same treatment applies to column flanges, which can be made far stronger for a given weight of metal by using vertical reinforcing brackets than by merely thickening the flange. In this case the designer should bear in mind that from ¼ to ⅜ in. must be taken off the flange in machining it, and proportion it accordingly.

The iron in Mr. Parsons' sample appears to be very high in silicon, which would also tend to the formation of shrink-holes. This metalloid should not exceed 2.25% in column castings.

The condition of the column flanges in the Darlington Hotel, as reported by Mr. Parsons, emphasizes the necessity of more technical knowledge on the part of designers and more care in selecting foundries where proper attention is paid to the qualities of the metal used to make structural castings.

Mr. O'Brien.

J. H. O'Brien, Assoc. M. Am. Soc. C. E.—The speaker would like to emphasize a point made by Mr. Schneider, in regard to skeleton-constructed buildings, namely, that the skeleton should be designed so that it will carry all loads and resist all external forces likely to come upon it at any time without assistance from the materials which clothe the skeleton.

The speaker will admit that this principle could not be carried out as effectively with a skeleton, the columns of which are cast iron, as with a skeleton constructed of steel throughout. But, as the previous speakers have disposed of the cast-iron column as unsafe for use in tall buildings, which, usually, are the most important skeleton structures, it may not be amiss at this time to touch upon the need of great care in the design of all important skeleton-constructed buildings.

The speaker has watched the construction of many such buildings, in which the framing is rectangular throughout, and as they grow, tier on tier, without diagonal bracing, or even large gusset plates to stiffen the joints, he has been impressed with their lack of native sta-

Mr. O'Brien. bility. In most of such cases, of course, the designer expects that external forces will be overcome by the aid of masonry curtain walls and floor systems; but, as these features are seldom erected simultaneously with the steel work, the risk involved, in depending on such assistance, is very great.

In fact, the speaker is aware of one such structure, at least, which was designed and erected by a reputable steel company (which had no control of the mason work), the roof of which collapsed, because it was loaded before the stiffening walls had been built up to reinforce the steel supports.

One of the first principles learned by engineers, who design structures, is that a rectangular frame has, theoretically, no stability, as forces applied along the members of such a frame will change its form. Therefore, the stability which attaches to structures of purely rectangular formation is furnished solely by the stiffness of their riveted connections, which are often designed improperly, usually only for the direct vertical loads.

The columns of many skeleton buildings are so placed, and are designed of such section, that many, and sometimes all, of their loads are eccentric (on the same side of the column); and, too often, no particular attention is paid to this eccentric loading, the effect of which should be offset by adding section to the columns and making sufficient joints at the connections.

The speaker hopes that Mr. Schneider's point, that skeleton structures should be designed to support all the loads, and to resist all external forces coming upon them, independent of the masonry, will be carried out more faithfully by designing engineers henceforth.

Mr. Just. GEORGE A. JUST, M. AM. SOC. C. E. (by letter).—Faulty design is a sufficient cause for the collapse of the building under discussion. Therefore, it is unnecessary to look for a reason, other than the inherent structural weakness incident to such faulty design.

This being so, it is eminently proper to inquire how the partial erection of such a structure could be possible, in the foremost city of the country under a code especially enacted to safeguard the public interests, as far as they are involved in the construction of buildings.

At the very beginning, a clear distinction should be made between the law, on the one hand, and the administration and interpretation of the law, on the other. The most perfect law, when administered or interpreted by incompetent or corrupt authority, proves ineffectual and often oppressive. But building laws, like all other civil laws, even when honestly administered, need intelligent interpretation and consequent modification from time to time, and it is remarkable that, while other branches of law are interpreted and modified by lawyers through the machinery of the courts, public sentiment does not demand that its laws relating to construction be interpreted by engineers, who, it must be admitted, are alone capable of directing the proper application of what is, after all, natural law, as against the law of accumulated precedent. Mr. Just.

This inconsistency, however, can be traced to an apparent, if not a real, lack of interest in public affairs by engineers in the past. They, consequently, lack weight when new thought is moulded, even when it relates to matters so distinctly within the engineering field as the formation or amendment of a "building-law."

Now the fact cannot be overlooked that a "building-law" must necessarily be perfectly general in its application; that all its provisions cannot possibly be made so comprehensive as to apply to each specific case, and that any attempt to do so must result in a limitation of individual rights, on the one hand, or endanger public safety, on the other.

And so it may be said that the best criticism made of the New York code, under which the collapsed structure had been partially erected, is that it "pretends too much engineering." The chance of repetition of a "Darlington" disaster would be materially reduced, if much of the present detail matter were eliminated, thus making the provisions of the code more general. The unprincipled practitioner would then not be able, as now, to force a Department of a municipal government to act practically as engineer in the design of work for which he himself is incompetent. It might terminate the practice of designing "according to law," and kill the assumption that individual structural members, when of proper section for the performance of their intended functions, will make a safe structure, irrespective of the manner of their assemblage or the quality of the general design. The result should be that experienced professional talent would be engaged more generally.

As to cast iron: The consensus of opinion, to-day, is undoubtedly favorable to a limitation of its uses, and it should be eliminated from structures of the character of the one under discussion; but even the characteristics and variable quality of cast iron do not call for its utter condemnation. Violent poisons, harmless in the hands of the chemist, prove dangerous playthings for children, and so with cast iron when in the hands of incompetents.

It may be conceded that, in the light of experiment, the permissible unit values, in the New York code, for cast iron in column form, are too liberal. These matters are often the result of compromise dictated by rival trade interests, strongly intrenched. Engineers, however, are under no compulsion to use such values. It is unfortunate, therefore, that when speculative ventures are involved, or the money for a project is limited, a marked tendency is shown to use the ambiguities and inconsistencies of the law to make the work "cheap."

Mr. Just. To this straining for a "cheap" building is also due the present custom of using cast columns in the lower, and steel sections in the upper, stories, with an utter disregard for possible initial bends, eccentric loading or proper transmission. This results in flimsy steel members, which have become an easy target for criticism by those who advocate the continued use of cast-iron columns.

And here it may be noted that the lawyer who avails himself of flaws in the civil and criminal law, to the advantage of his client, enhances his reputation, but the engineer who violates the laws of statics, not only impairs his reputation, but insures the failure of his work.

In recurring to the question of the improper interpretation of the spirit of a law, it can be positively asserted that it was not the intention, of the framers of the New York code, to permit the erection of hybrid structures of the "Darlington" type, and if the decision to the contrary, which has created such a bad precedent, is irrevocable, then the law cannot be too promptly modified in this particular.

It is remarkable how the efforts of the framers were nullified, in this particular. Recognizing the then existing tendency to depart from a pure skeleton type in which the frame should be at all times self-supporting, they intended to recognize only two types, (1) the old, wall-bearing type, and (2) the modern skeleton-frame type of building.

To distinguish the two types clearly, they demanded for the first time that "wall-girders" be placed continuously at all floor levels in skeleton-frame constructions (Sec. 110). They reduced the then existing minimum thickness of walls for this type (Sec. 36), and removed the heretofore general restriction that front and side walls must be carried up in certain relation to each other, and confined this requirement to wall-bearing buildings (Sec. 41).

No added rigidity, from floor filling or arches, was to be relied upon, for the requirement that the progress of filling in, or floor arching, should bear a certain relation to the height of the frame is not now, and never was, a part of the New York building code. This is a state labor law, enacted presumably for the better protection of workmen engaged on buildings.* The new code also, for the first time, distinctly empowered the head of the Department to call for such structural details as in his judgment might be necessary (Sec. 4).

Therefore, it is unfortunate for good construction that, as was testified by an engineer of the building bureau, before the Coroner's Jury, a joint reading of Sections 36 and 37 was interpreted as a permit to erect buildings of the "Darlington" type. This precipitated upon the public an incomplete and insecure frame, without the restriction

* Chap. 32, Gen. Laws, Art. 1—Sec. 20, as amended by Chap. 192—laws of 1899.

that the enclosure walls must be carried up at the same time—removing the only element which might possibly have prevented collapse, as it no doubt has in many cases of equally bad design. The curtain walls named in Sec. 37 were not, in the minds of the framers of the code, intended to include any exterior walls, but referred to non-bearing, interior division walls only. Mr. Just.

The present law is weak, in so far as it lacks requirements for proper superintendence, and the "regulations" of the Department on this point are no better. This could be remedied by an amendment requiring that: No building shall hereafter be erected without the continuous direction and superintendence of an architect, or civil engineer, who shall have had at least five years' experience in building construction; no work shall be commenced until the name and address of such superintendent is filed with the Bureau of Buildings, and all changes of superintendent during construction shall be similarly certified to said bureau.

The influence of the American Institute of Architects could be very properly exerted for the enactment of such a section. It would tend to break up the present pernicious practice of architects engaged on speculative work, who accept commissions for the making and filing of plans, leaving the execution of the work to incompetent or unscrupulous persons.

Too often, this early termination of relations between architect and client, is only apparent, and at times results in deceiving the public authorities. Proof of such continued relationship, however, is shown when these architects, as is often the case, permit themselves to be used to harass the contractors, for the benefit of their speculative employers.

Vol. XXX. AUGUST, 1904. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

A PHENOMENAL LAND SLIDE.

Discussion.*

By James D. Schuyler, M. Am. Soc. C. E.

Mr. Schuyler. James D. Schuyler, M. Am. Soc. C. E. (by letter).—An opportunity to add a few words to the discussion of this admirable paper is eagerly embraced, although protracted absence from home on professional work, out of touch with the Society literature, had almost prevented the writer from participating in a subject of deepest personal interest to him.

During 1894, the writer, with the title of "Consulting Engineer," was employed to design and construct the two reservoirs, Nos 3 and 4, involved in this landslide, as well as two others, Nos. 1 and 2, on the east side of the river.

The late Isaac W. Smith, M. Am. Soc. C. E., was the Chief Engineer of the Water-Works, and Mr. Clarke was his principal Assistant, in charge of the construction of the main pipe line from Bull Run to Portland.

The construction of the four reservoirs, in the few short months of fair weather available in the moist Oregon climate, was a most herculean task, and, under ordinary conditions, should have occupied at least two seasons, but, as the realization of a pure water supply came nearer, there was great public clamor for the completion of the works before the close of 1894, and as the contractors for the pipe making and laying needed to be stimulated to extra exertion by the evidence of preparedness on the part of the city's engineer to receive water

* Continued from May, 1904, *Proceedings*. See March, 1904, *Proceedings*, for paper on this subject by D. D. Clarke, M. Am. Soc. C. E.

through the pipe; and, furthermore, as a large money saving, amounting to $100 000 or more, was to be made by eliminating the cost of pumping the city's water supply by an early completion of the system, there was a strong incentive for pushing work on the reservoirs as rapidly as possible. Mr. Schuyler.

The writer's judgment inclined to more deliberate action, but the public eagerness to enjoy Bull Run water at the earliest possible moment, and the possible saving in cost of operation seemed sufficient inducement to overcome his conservatism in this regard.

Delay in completing the reservoir until 1895 would have given the slide opportunity to develop without destroying the linings, but would probably have led to the abandonment of the reservoir sites. This would have been a misfortune, for the reason that these reservoirs are in the most convenient locality possible, and they can yet be made available for reservoir purposes by a thorough drainage of the slide. Any substitution of them, by the construction of reservoirs of equal capacity at corresponding elevations elsewhere, would have been unquestionably more expensive to the city than the losses involved in interest on the cost of the unused portion of the work and the cost of that part of the reservoir linings which has been shattered and destroyed by the movement; added to the subsequent expenditure made in exploring and draining the slide. Events have proven that the city has really been a gainer by what has occurred, rather than otherwise.

It may be considered as particularly fortunate that the gate-houses and pipe connections were designed in such a careful manner that the breaks in the reservoirs had in no wise interrupted the service, and the works are as efficient as though the reservoirs had never been injured by the slide. Even the pumping plant, operated by the water falling from Reservoir No. 3 to Reservoir No. 4, was not stopped a day in its operation of the high-level pumps.

The developments, described so minutely and interestingly in the paper, of the extent, depth, area and volume of this remarkable slide, the very slow rate of its movement, and the effect produced upon it by drainage, constitute an impressive array of facts teaching a lesson of the importance of far-reaching investigation prior to beginning important construction. The preliminary investigations and borings, referred to in the paper, which were made prior to the engagement of the writer, were reviewed carefully by him at the beginning of the work, and appeared to be conclusive and satisfactory in demonstrating that the foundations of the reservoirs were entire stable, and that the numerous local land slips, appearing on all sides of the reservoir sites, were not deep-seated, and would be cut out by the excavations planned. The writer examined carefully all the hill slopes above the reservoir many times during the excavation of the basins, and never saw the slightest indication of the general slide such as developed subse-

Mr. Schuyler. quently. As stated, the small surface slips were numerous, not only on the west side of both reservoirs, but on their east, north and south slopes. They were caused evidently by super-saturation of the soil, standing on slopes steeper than the natural angle of repose, and the manifest remedy was drainage. To accomplish this drainage in the most efficient way possible so that the linings might not be disturbed when the reservoirs were finally in service, an elaborate system of slope and bottom drainage was planned and carried out, consisting in part of numerous drive-well points driven into the slope, to relieve the water as far back of the surface as possible. These were connected with tile drains laid up and down the slope at frequent intervals, in trenches filled with broken stone. These tile drains discharged into sewer pipes laid at the toe of the slope, underneath the lining, all around the reservoir, emptying into the city sewers outside the dam.

The most extensive and troublesome of these surface slips was the one which constantly discharged mud into the excavations of Reservoir No. 4, about on the line of the cable road. This slide extended west almost as far as the curve, the distortion of which is shown in Fig. 2, Plate VI, and was lubricated constantly by spring water following down the cable road track. The writer noticed the distorted rails at this curve on several occasions during the season, and ascribed their condition to the local settlement, and the slide below it.

From all the phenomena observed by the writer during the construction of the reservoirs, he is of the opinion that the large, main slide, the outlines of which were revealed in the year following the excavations of the reservoirs, was in a state of rest throughout the year 1894, up to the time when the cracks first appeared in the west slope of Reservoir No. 4, in August. The protrusion of the upper layer of clay in a marked line, and at a rapid rate, out beyond the face of the excavation was sudden and startling, and although at the time it appeared to be but a deeper manifestation of the old troublesome cable road slip than had yet been apparent, it is now clear that it was the beginning of the movement of the big, main slide. That this was caused by the removal of the toe of the slide in the excavation of the reservoirs is indicated chiefly by the fact that it occurred in the dry season and at the latter end of the summer. Had the slide been moving during all the year previous, the manifestation of it could not have been overlooked. Evidently, it was in a state of unstable equilibrium requiring but small cause to start moving again, and, although the volume of excavation removed from the reservoirs was but 3% of that of the moving area, as pointed out by the author, it was the key which held the mass from continuing its ancient movement.

All other great slides of this region have their greatest movement during the rainy season, while this one apparently started in August, the driest part of the year, with a movement of ½ in. per day for several days, although the subsequent maximum rate was but 12 in. per annum. Mr. Schuyler.

During the progress of this work, in the early part of 1894, a land slide occurred at Portland Heights, about half a mile southeast of Reservoir No. 4, which suddenly overwhelmed a valuable dwelling with mud, and destroyed its contents. The volume of earth which slid was probably less than 5 000 cu. yd., originating in a street embankment on the heights near the edge of a high bluff. It descended a slope steeper than 1 on 1, a vertical height of from 300 to 400 ft., and, after starting, its movement was rather in the nature of an avalanche of mud than that of an ordinary land slide. It was evidently something of this character which was expected by the Committee of 100, referred to in the paper, when they protested against the location of the reservoirs in the City Park, as the Portland Heights slide was of very recent occurrence and fresh in the mind of everyone when they made their protest. Such a torrential movement alone could have displaced the water in the reservoirs and caused a sudden flood, and it was against an accident of this type that the Committee expressed their fears. The Engineers of the city saw no evidence of the possibility of a slide of that character in the vicinity, and so favored the continuance of work. They were at least as well able to judge as the Committee of 100 novices and laymen, and they certainly did not see a sign of the deep-seated ancient slide, the slow movement of which afterward put the reservoirs out of service.

That this slide was caused primarily by water and the lubrication which water afforded to the under surface of the moving mass, is so manifest as to be axiomatic. Practically all land slides have the same moving cause, and would remain stable but for water.

The most notable and extensive land slides which have come under the writer's observation occurred on the line of the Canadian Pacific Railway, a few years ago, as the result of irrigating high bench-lands along the Fraser River. The litigation and investigation which followed was described by the writer in a paper read before the National Irrigation Congress of 1895, held at Phœnix, Arizona. The cause of these slides was similar to those which developed the slide under discussion, *viz.*, removal of the toe of the old slide, in a state of rest, by a railway cut, and the continued application of water for irrigation, in the same saturating excess which produced the slide before the railway was built. Irrigation caused the slide; after the original movement it reached a state of equilibrium where it was no longer moving, during which period the railway location was made; subsequently, the railway cutting removed the toe of the slide and started its movement afresh, and continued irrigation kept the ground lubricated and in motion until the courts enjoined the irrigation, and the trouble ceased.

Vol. XXX. AUGUST, 1904. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

ON SEDIMENTATION.

Discussion.*

By Messrs. Galen W. Pearsons and Robert Spurr Weston.

Mr. Pearsons. Galen W. Pearsons, M. Am. Soc. C. E. (by letter).—It is probably correct to say that the waters of our western rivers have been treated more successfully by sedimentation than by filtration, but neither, alone, has been fully satisfactory. Their constantly changing condition, as to high and low water, and the sources from which this water is derived, make the problem a very difficult one. While, most of the year, they may be treated successfully, there are times when they are very obstinate.

At Memphis the writer observed a railroad tank, filled with water from the Wolf River; this, standing unused for three months, maintained its dark color, and, on being emptied, showed no appreciable sediment.

The Kansas River, sometimes for weeks together, shows a really human aversion to settling, and gives the Missouri a similar character. Above Kansas City, the Missouri is not so obstinate; it has bad spells, but not such long ones; the large amount of sediment precludes filtration without previous sedimentation, and, except in some of the smaller towns, the water is little used.

It is stated † that the Missouri carries a maximum of 1 part by

* This discussion (of the paper by Allen Hazen, M. Am. Soc. C. E., printed in *Proceedings* for April, 1904), is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. Communications on this subject received prior to September 23d, 1904, will be published subsequently.

† *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 239.

weight of sediment in 673. That amount is often exceeded at Kansas City. Mr. Pearsons.

In designing the settling basins for Kansas City, in 1874, the writer had the benefit of the experience of the late Henry Flad and T. J. Whitman, Members, Am. Soc. C. E., of St. Louis, and of the late Birdsill Holly, of Lockport, N. Y., who had made careful experiments on both sedimentation and filtration.

The sedimentation at St. Louis was then alternate; Colonel Flad's experiments showed that in still water, of 1 000 parts of sediment, 820 subsided in 6 hours, 900 parts in 12 hours, 930 parts in 18 hours, 945 parts in 24 hours, 966 parts in 48 hours, 969 parts in 96 hours. This was when the condition of the river was favorable for subsidence. In the writer's experiments he seldom found such rapid results, but considers the proportions of sediment carried down in 1, 2, 3, 4, 8 and 16 units of time, as a close approximation.

Mr. Holly's experiments showed that water could be moved at a rate not exceeding 1 ft. per minute, care being taken to make the flow uniform, with but little loss of efficiency in subsidence; and the need of a uniform water level decided the construction of the basins for continuous flow, although their shape was not favorable for the best results. The writer's experience with these has led him to consider continuous flow preferable to intermittent.

While it may be necessary to consider the particles forming the sediment as of uniform hydraulic value, to enable a beginning of mathematical deduction, such uniformity does not exist; observations of the dust falling on a microscope slide will show some particles one hundred times as large as others, and the same, no doubt, is true of the sediment in rivers.

Experiments made by the writer, before designing the settling Kansas City basins, gave results which may be useful in the consideration of the subject.

Clear, flint glass tubes, 3 ins. in internal diameter, were joined with heavy rubber bands, making one tube 5 ft. high, one of 10 ft. and one of 15 ft. These were placed before a tall window, side by side, with the bottoms of the three tubes level, and filled alternately with water from the Kansas and Missouri Rivers, each time filling the three tubes together. Mr. Holly observed some of the experiments. His first observation was that the water cleared quickly at the top, as has been claimed so generally, but, on covering the tops of the tubes, this was seen to be only an appearance caused by light falling on the surface of the water, and no difference in color could be seen between the top and bottom of any of the tubes at any time during the subsidence of the water, the bottom of the 15-ft. tube showing no more color than the 5-ft. tube beside it, at any stage of the subsidence.

This was unexpected, and careful examination was made to dis-

Mr. Pearsons. cover the reason. At first filling, the water was so turbid that it only showed color. As it cleared gradually the writer was able to see particles descending near the bottom of the 15-ft. tube; at times, something of the same could be discerned in the 10-ft. tube, but with difficulty, and none at any time in the 5-ft. tube.

These particles, by their uniform shape, explained their origin and action; they were pear-shaped, or rather like little tadpoles swimming head down, the tails tapering to invisibility; plainly some larger particle by its quicker descent had overtaken and joined smaller ones, and, increasing by constant addition, had at last become visible, their motion near the bottom being so rapid that, if it had been uniform, but a few minutes would have been required for the whole descent.

These observations induced the belief that more depth conld be used in sedimentation than had been considered advantageous, thus increasing the cross-section by depth instead of area. The fragile character of these aggregated particles of soft mud also showed how necessary stillness was to guard against their disruption, and fully bears out all that has been claimed for the use and necessity of baffle partitions.

In 1886 the source of supply was changed from the Kansas to the Missouri River, and continuous flow provided for in the new basins, though their shape made its use less effective than if they had been parallelograms. The water was received in a central basin, holding some 8 hours' supply, and provided with a bottom having a slope of 10% to admit frequent and rapid cleaning. From this the water passed through three other basins, each having a capacity of about 1 day's supply (when the flow was from 12 to 14 millions of gallons per day). The receiving basin gave deposits aggregating as much as 30 ft. in depth per year, the second basin about one-third of this depth, and the last only a few inches. Of this deposit, nearly one-half was so thin and soft that it would flow out without washing, and there was no indication in any of the basins of any critical bottom velocity.

The writer dissents entirely from the idea in Proposition 12, that any instantaneous clearing of the surface exists. Though the water at the surface may naturally be considered as freed more quickly from the heavier sediment, its color will be kept to a considerable extent by the lighter material left behind, and the clearing be found more apparent than real. This relates rather to color than weight of sediment, which is doubtless less near the surface.

In Proposition 13, the idea is given that the particles of different hydraulic value simply follow each its own rate of descent, whereas the writer's experiments show that they coalesce with other particles and increase in size as they descend. Although this increase may be only mechanical, it would seem, even when no coagulants are used, that the action may be analagous to that of the formation of rain drops.

Of filtration, Colonel Flad's experiments showed that upward filtration was effective fourteen times as long as downward, with the same water and rate of flow; that is, it required fourteen times as long to clog the filter. This may be considered properly in connection with the subject, for the reason that the upward movement of the water was not greatly in excess of the downward movement of the sediment, and that the flow was under ideal conditions as to stillness in the water treated. Such filtration, in connection with adequate sedimentation, would be a step in advance of any present treatment. This is speaking of the subject in a mechanical light; other considerations may modify these statements, which are not intended as a criticism of Mr. Hazen's able paper, but as presenting some data which may be of service. Mr. Pearsons.

ROBERT SPURR WESTON, ASSOC. M. AM. SOC. C. E.—At New Orleans the speaker had some experience with small subsiding basins—basins 50 ft. long and 10 ft. deep—and, while the experience gained is not in any way opposed to the formulas, assumptions and propositions in Mr. Weston.

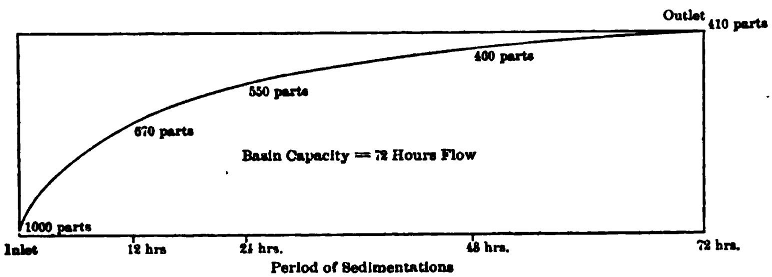


FIG. 2.

this excellent paper, at many times results were obtained which did not agree with those obtained by Sedden in St. Louis. These opposite results, however, may be explained, perhaps, by differences in local conditions, such as the relative temperatures of air and water.

Much of the Mississippi River water at New Orleans, when introduced into subsiding basins, stratified quickly, especially during the periods when the temperature of the air was higher than that of the water. The incoming turbid water appeared to flow under the water already in the basin, and, leaving part of the suspended matter behind, appeared to rise vertically to the surface layers before it moved toward the outlet.

The inlets of the basins were at one end, 2 ft. from the bottom, and the outlets—overflow weirs the full width of the basin—were at the other. One might argue from certain premises that in these basins the curve of the suspended matter remaining after different periods would reach from inlet to outlet along a curve approaching a parabola, as illustrated by the diagram, Fig. 2.

Mr. Weston. At times such conditions were not observed, however, as Table 4* will show.

TABLE 4.—Variations of Turbidity and Temperature of Water in Subsiding Basins at Various Depths.

Date. 1901.	Hour.	Source.	Depth, in feet.	River Water. Silica turbidity.	River Water. Temperature, in degrees, Fahr.	Inlet End. Silica turbidity.	Inlet End. Temperature, in degrees, Fahr.	Outlet End. Silica turbidity.	Outlet End. Temperature, in degrees, Fahr.
Mar. 26..	10	S. B. 1	0	900	52.1	10	62.6	20	64.4
" " ..	"	" "	2	900	52.1	12	61.7	25	61.7
" " ..	"	" "	4	900	52.1	190	56.8	220	56.3
" " ..	"	" "	6	900	52.1	380	56.8	400	56.3
" " ..	"	" "	8	900	52.1	480	55.4	440	55.4
" 27..	9	" "	0	950	54.6	40	64.4	80	63.5
" " ..	"	" "	2	950	54.6	180	58.1	180	58.1
" " ..	"	" "	4	950	54.6	360	55.4	360	56.3
" " ..	"	" "	6	950	54.6	500	55.4	550	55.0
" " ..	"	" "	8	950	54.6	625	54.6	600	55.0
" 28..	10	" "	0	975	53.2	100	....	100	....
" " ..	"	" "	2	975	53.2	270	....	270	....
" " ..	"	" "	4	975	53.2	400	....	380	....
" " ..	"	" "	6	975	53.2	440	....	480	....
" " ..	"	" "	8	975	53.2	600	....	600	....

Note: Silica turbidity results are given in parts per million.

In Table 4 the increasing temperature of the basin water, during storage, should be noted. This is more of a factor in the South, perhaps, than in other localities, where the temperature of the air would be higher than that of the water for a shorter period during any average year.

At other seasons of the year, at New Orleans, when the temperature of the river water was not as low as that of the air, such differences in turbidity, between the samples taken at different depths, could not be observed.

Baffles are important aids to sedimentation, and, whereas excellent results in subsiding basins are obtained when conditions promote stratification, the speaker believes that the ideal baffle is one which similates stratification, or, as Mr. Hazen has said, a horizontal baffle. In reference to this, it may be interesting to mention the fact that, in water-softening plants, highly increased efficiencies of sedimentation have been obtained by a modification of this design, that is, inclined baffles, inclined so that they may be cleaned.

A trade, water-softening tank, in common use, is an example of this form of construction. In this device the inclined perforated plates are arranged around the wall of the tall subsiding tank; these plates are like shelves one above another. They incline toward the

* "Report on Water and Sewerage," New Orleans, 1903.

center, which is open. Whenever necessary, the accumulation on the plates is washed to the center of the tank, where it falls to the bottom, whence it may be readily washed out. Mr. Weston.

But it is one thing to remove successfully the coarse crystalline sediment resulting from the softening process, and another to remove by such a process the fine colloidal clay particles from natural waters.

It seems to the speaker that the ideal condition for economical subsidence is that outlined under Proposition 12, in the paper, where the water enters near the bottom of the tank and escapes near the top. The nearest approach to this condition would seem to be basins with frequent baffles. These baffles should be built so as to skim the water from the first compartment and deliver it near the bottom of the following compartment. One reason for this is that the frequent bringing of the water to the bottom of the basin, provided always that the velocity is not increased greatly thereby, diminishes the distance through which the particles must sink. It is easier to prevent particles from rising than to precipitate these same particles against the same current.

With this form of construction, it would be easy to arrange to use the whole basin when conditions were such that stratification would exist, and, if it did not exist, the sedimentation basins would be broken up into compartments, thereby diminishing the effect of wind and temperature disturbances.

Mr. Hazen bases a large number of propositions on one value of t. It would be interesting to substitute other values of t,—for instance, infinity. In the latter case, t would be a quantity vastly different from those mentioned in the paper. Mr. Fuller has illustrated this phase of the conception very clearly by noting that part of the colloidal suspended matter, particularly the fine clay in the rivers of the Mississippi Basin, never settles, that is, never from a practical standpoint. This clay is so fine, however, that it is beyond the reach of plain subsidence, and should be treated in other ways. Continuing with this thought along the line of Mr. Hazen's conclusions, one would come to believe that multifold filtration, leaving out of consideration the absorptive power of the sand itself, would not remove continuously and completely the last particles of fine clay, except at prohibitively low rates of filtration.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE GATUN DAM.

Discussion.*

By Charles L. Harrison, M. Am. Soc. C. E.

Mr. Harrison. Charles L. Harrison, M. Am. Soc. C. E.—Mr. Ward's suggestion that the dam and locks for controlling the waters of the Chagres River and Panama Canal on the Atlantic side be built at Gatun instead of Bohio, is presented for consideration and not as a final project. In the absence of definite information as to the foundations, no attempt is made to give a detailed design, nor to make an accurate estimate of cost. For the purpose of giving an approximation to the cost, the author has made a comparison with the dam at Bohio, as proposed by the Isthmian Canal Commission, which is perhaps the best that can be done in that direction, in the light of present information on the subject. It is very unsatisfactory for the purposes of this discussion to have so little information concerning the character and extent of the material overlying the bed-rock at the proposed location of the dam. As far as the speaker knows, no borings have been made on any part of this line for the purpose of determining the elevation of the bed-rock. However, some comparatively shallow excavations have been made in this vicinity in excavating the Panama Canal and the Diversions for the Gatuncillo and Chagres Rivers.

Starting at the Pacific Coast, near the City of Panama, the general line of the canal runs in a northerly direction to the Atlantic near

* This discussion (of the paper by C. D. Ward, M. Am. Soc. C. E., printed in *Proceedings* for April, 1904), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Communications on this subject received prior to September 23d, 1904, will be published subsequently.

Colon; but, to avoid confusion in the directions, it will be considered as running north and south, and that part of the Isthmus extending toward South America as being to the east. The range of mountains between the two oceans, and approximately parallel to their coasts, is of volcanic origin. This range of mountains—of moderate height for the entire length of the Isthmus between North and South America—is the only regular feature of the formation. In the vicinity of the Panama Canal the surface of the ground was left in very irregular ridges, valleys and knolls or hills. The Caribbean Sea at one time extended over the entire area of what is now the alluvial swamps of the Chagres River, as far inland as Bohio, where it was connected through a narrow channel with a bay extending over the area approximating in outline the lake which would be formed after constructing the proposed Bohio Dam. To the east of Obispo there existed a large area with no direct outlet, north or south, to either the Atlantic or Pacific Oceans, and the drainage from it flowed west to Obispo and thence north in the lowest valley to the Atlantic. The heavy tropical rains, falling on soil unprotected by vegetation, evidently carried large quantities of silt toward the sea, and resulted in first filling up the valley between Obispo and Bohio and later forming the swamps between Bohio and the sea. This seems to be the probable geological history of the formation of the present Chagres River and the low lands adjacent to it. It is possible that there may have been considerable subsidence of the coastal territory, as seems fairly well established some distance westward along the Nicaragua Coast. In making the borings at the site of the Bohio Dam, pieces of timber were encountered at a depth of about 100 ft. below sea level, and, in one hole, a log of at least 1 ft. in diameter was found at this depth. Also, in places, sand to a depth of 100 ft. overlies the bed-rock. These data indicate that at one time the bed of the stream corresponded with the present rock surface, and that the entire deposit overlying it was formed in running water.

Mr. Harrison.

Between Bohio and the sea a great many knolls of rock project in the ancient valley. Some have their tops about level with the present swamps, others are many feet below it, and still others exist as rocky islands. With such an irregular formation, and in the absence of borings, the depth to rock between these knolls cannot be guessed with any degree of probability. In fact, some of the knolls which project above the swamp are not shown on the existing contour maps of this territory. This is not surprising when the great difficulties of making surveys in that country are known. The low lands are covered with a tropical growth so dense that it is impossible to see into it more than a few feet, and lines must be cut to every point where an elevation is to be taken. It is possible to run a line within 100 ft. of a hill from 30 to 40 ft. high and not discover its existence. Just

Mr. Harrison. south of the west end of the proposed Gatun Dam, Plate XXI, will be seen an excavation starting from the Chagres River. It is related that when this cut was begun it was intended to connect with a tangent of the cut north of the proposed Gatun Dam, and thus form a channel for diverting the waters of the Chagres; but, after the work had progressed for the distance from the river shown on the map, a rocky hill was discovered—the existence of which had not been known before—and the diversion channel was moved further down stream, as shown. A heavy wooded growth, not only makes surveys difficult and expensive, but conceals the irregularities of the ground surface.

Evidently, the existing contour maps were not intended to be made in that detail and extent necessary to give the information for locating a dam to be built to a height of 10J ft. above sea level in the neighborhood of Gatun. It is possible that careful surveys would develop the necessity of building dams or dikes, other than the one shown on Plate XXI, to impound water at Elevation 90 in the lake. The existence of a proper location for a spillway to discharge the flood waters of the Charges is a very important item. If located near the east end of the dam, it would be expensive and would discharge the water into the canal below the dam, unless this were obviated by constructing for it a separate diversion channel to the sea. A small channel has already been excavated for diverting the waters of the Rio Gatuncillo and the Mindi, but it would have to be enlarged very much to take the flood waters of the Chagres. By locating the spillway near the west end of the dam, the floods would discharge through the Chagres into the sea without interfering with the canal, but a good location for it is not shown on the maps.

With existing data as to the contour of the country and the character of the foundations, it seems to be impossible to make an estimate of the cost ot the Gatun Dam, or to make a satisfactory design. For the purpose of making the estimate of cost, the author has compared its length with that of the dam at Bohio proposed by the Isthmian Canal Commission. This may not be a proper comparison. The length of the Gatun Dam may be greater than estimated, and the depth to rock foundation is an unknown quantity. In the paper* on "The Bohio Dam," by the late George S. Morison, Past-President, Am. Soc. C. E., is given, in Fig. 2, the location of several sections where borings were made. The cross-sections of the valley, on these lines, are shown in Fig. 3. It will be seen that Sections G, B, C and D are all shorter than Section F, and that the depth of rock on Section F is less than on any of the others. In the last-named location the greatest depth to rock is west of the river, and not under its present bed, which, for reasons that need not be given in detail here, was considered a favorable circumstance. Although the depth, even here, was greater than

* *Transactions*, Am. Soc. C. E., Vol. XLVIII, p. 235.

at any other place where foundations of magnitude have been attempted heretofore by the pneumatic process, the adoption of this location for this purpose seemed advisable. The Isthmian Canal Commission, therefore, selected a location where the bed-rock was highest, though it gave a greater length of dam. Doubtless, it was of the opinion that the cost of construction on this line would be less per linear foot than on any of the other lines, and the hazards of construction much less. In view of the unknown depth to good foundations at Gatun, the cost of that dam per linear foot may differ very markedly from the one at the Bohio location. That the cost would be very great, there is no doubt, but the data are not sufficient to make even an approximate estimate. Mr. Harrison.

It is suggested that in case the 90-ft. dam is thought inadvisable, one to impound the water at Elevation 45 might be built. This is open to the same objection as the 90-ft. dam, though the risk might not be as great.

"This elevation is suggested, as 45 ft. seems to be the maximum permissible lift for a lock." Why this is so is not clear. The lock walls can be built just as strong for a 50-ft. lift as for a 45-ft. lift. The uncertainty, if any, must then exist in the lock gates. These would be of metal, and could as well be designed to withstand the water pressure due to a 50-ft. head as for that due to a 45-ft. head. A bridge of 100-ft. span can be designed to carry a load of 100 lbs. per square foot, and one of 500-ft. span can be designed to carry the same load. One would not be considered less safe than the other under this loading. The same reasoning can be applied to lock gates. Under some conditions, the quantity of water used at each lockage might be a controlling factor in determining the lift. If it is assumed that the gates will be wrecked by vessels: such accidents are as likely to happen to one as to the other. The loss of water from the reservoir, when the lift of the lock is greater than the depth of the channel, will be the same in each case, as it is controlled by the elevation of the upper miter sill. It is believed that the lift of the lock need not limit to 45 ft. the height of the alternate dam. The Board of Engineers on Deep Waterways, in June, 1900, reported on a ship canal from the Great Lakes to the Atlantic tide waters, and recommended a lock of 52-ft. lift on that part of the canal between the St. Lawrence River and Lake Champlain. There seems to be no reason why it could not be built and operated successfully.

It is also possible that the advantages, in navigating a broad, shallow channel, over those of a restricted, well-defined channel, are over-estimated.

In all the studies for a high-level canal at Panama, the dam on the Atlantic slope has presented the greatest difficulties, and has seemed to be recognized as the least safe structure in the entire project. If

Mr. Harrison. this view is correct, it would be prudent not to increase these uncertainties by building the dam unnecessarily long.

A study of the contour maps and an inspection of the country along the route of the canal show that the narrowest point in the valley is in the vicinity of the proposed Bohio Dam. If the dam were located above this, it would be in a wide basin; and, if below, it would be in the broad swamps of the Chagres.

It is intended in these remarks only to point out more fully the physical conditions, as far as they are known, at the two sites, Bohio and Gatun. It is conceivable, although extremely improbable, that further surface and sub-surface examinations at the Gatun location may reveal conditions making the comparison more favorable to it than the present information would indicate.

The dam proposed at La Boca, near the Pacific terminus of the canal, seems even less advisable than the Gatun Dam.

Vol. XXX. AUGUST, 1904. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

LATERAL EARTH PRESSURES AND RELATED PHENOMENA.

Discussion.*

By Messrs. Robert Brewster Stanton and Richard Lamb.

Robert Brewster Stanton, M. Am. Soc. C. E.—The subject of "Lateral Earth Pressures" has been one of great interest to the speaker for the past thirty years, especially in connection with the phenomena of great land slides, both natural and artificial, with which he has been specially connected, beginning with the construction of the Cincinnati Southern Railway in Tennessee in 1874. Mr. Stanton.

The author of the paper is to be congratulated upon his careful and painstaking study of the subject, from the technical and mathematical side of the question; and all engineers engaged in public works, where such conditions are encountered, owe him and other mathematical investigators a debt of gratitude.

Every engineer who has made a study of this subject, and has had occasion to apply the same in practice, knows full well the great value to the profession of all such scientific discussions, and the mathematical formulas deduced from experiments in the laboratory. The speaker has no intention whatever of casting any reflection upon such mathematical investigations, either pure or applied, for in years gone by he himself has been somewhat of a mathematical gymnast, but he desires to call attention to one or two very important facts, which,

* Continued from May, 1904, *Proceedings*. See March, 1904, *Proceedings*, for paper on this subject by E. P. Goodrich, Jun. Am. Soc. C. E.

Mr. Stanton. it would seem, are almost entirely omitted in investigating lateral earth pressures and land-slide phenomena.

More than thirty years' experience in contact with actual conditions in Nature, throughout the Rocky Mountain region from Canada to Mexico, has convinced the speaker that, in the construction of retaining walls, and other works to resist lateral earth pressures, and in the investigation of the causes of great land slides, two most important conditions are usually overlooked.

The mathematical formulas are absolutely correct, as deduced from the conditions, as stated; but, how often are just such conditions found on the mountain side, in the reservoir, or in the great land slides of the world? The mechanical conditions are there, and the mathematically calculated results should occur; but, how often are the beautiful formulas entirely upset and made utterly useless by Nature quietly inserting some little geological or chemical coefficient which has been entirely neglected in the calculations?

Hence, the point to which it is intended to draw particular attention, is the dependence of every such formula and all such investigations upon the "interdependence of all sciences, as applied to engineering works."

In the discussion of lateral earth pressures and the causes and effects of great natural land slides which produce such tremendous lateral pressures, two other sciences, namely, geology and chemistry, are absolutely necessary—in addition to the mathematics—and perhaps some others also.

About eight years ago the speaker was employed by the Canadian Pacific Railway Company, to examine and report upon the great land slides on the Thompson River, in British Columbia, and it is intended now to give only a few facts from the results of that investigation, to illustrate the point in hand, for the reason that some, at least, may not find time to read a paper* by the speaker on this subject in which the peculiar conditions encountered and the methods of investigation are described at some length.

To be very brief: The slides referred to occurred at a point on the Canadian Pacific Railway about 200 miles east of Vancouver, B. C., near where the railway passes through the Black Cañon Tunnel. This portion of the Thompson River traverses a gorge about 5 miles wide at the top, and about 2 000 ft. deep, with hills and higher ranges rising back on each side to elevations between 5 000 and 7 000 ft. The river runs in an inner gorge. There is but little bottom land near the river, the real banks of which are from 50 to 150 ft. above low water, and which extend upward in benches and terraces back from the river and to much greater heights.

* "The Great Land-Slides on the Canadian Pacific Railway in British Columbia." By Robert Brewster Stanton, M. Inst. C. E., *Minutes of Proceedings*, Inst. C. E., Vol. CXXXII, p. 1.

Up to the time of the speaker's examination it had been believed that the great slides were caused by a seam of clay on bed-rock, similar to the Portland slides, and so common in most such land slides or slips. The water from the irrigation of farming lands on the benches above (which had always been understood as the direct cause of the slides) was supposed to have sunk down through the underlying material, and, lubricating the stratum of clay on bed-rock, destroyed its friction, and enabled the mass to slip down an incline into the river, carrying the railroad embankments and cuts with it. Mr. Stanton.

As an explanation, it may be of interest to state that in this section of British Columbia, only 100 miles east of the Pacific Coast, where the rainfall at Vancouver attains 62 ins., and at other points even exceeds 150 ins. per annum, there is a country as hot and dry as Arizona, where not a blade of grass can be raised without irrigation, and the temperature ranges from 120° in summer to —50° in winter.

After a careful and extended examination of the slides and the country around them, the speaker was convinced that no such stratum of clay and no bed-rock sloping toward the river existed, at least above the bed of the river, so as to cause the results noted; and that, although the irrigation water was in fact the direct cause of the movements, the question was not one of mechanics or mathematics, but rather one of geology and chemistry, influenced perhaps by some remoter sciences.

There were eight separate portions affected, six being comparatively small, with two very important slips known as the "North" and "South" slides. At both places the country originally sloped up from the river in a series of benches or terraces to the first line of hills. The south slide has an extreme length of 1 880 ft. along the railway, and an extreme width, back from the river, of 1 575 ft. It is of somewhat irregular form, with a semicircular outline at the back, and covers an area of 66 acres. The north slide has a maximum width at its widest portion of nearly 0.5 mile, and a length, back from the river, approaching 0.75 mile, with the same semicircular back line. It is of irregular form, and covers an area of 155 acres. The height of the first bench next to the river, in both cases, was originally about 80 ft. above low-water level. The land then rose in successive levels to a height, on the south slide, of 400 ft. to the bench at the top, or back edge, where the cave-down broke off the solid ground, and, in the case of the north slide, it extended to the third higher bench 500 ft. above the river. It is impossible to ascertain at what depth these enormous masses of earth and loose rock broke, or, in other words, the depth of the plane on which the mass moved toward the river; but it is estimated that at the back edge of the south slide the break fell almost vertically for a distance of more than 300 ft., and on the north slide perhaps more than 400 ft.

Mr. Stanton. Speaking generally, after the great valley of the Thompson River was cut out by glacial action, it was filled again with glacial drift and silt, and once more the river cut its channel down through this glacial deposit and now flows in a narrow gorge, within the greater one, which slopes in benches and terraces up to the hills and mountains on both sides.

The terraces on each side of the valley along this section consist of the soil on the top of each bench of light sandy loam to the depth of from 1 ft. to 8 ft. Below, in places, is found from 3 to 10 ft. of clean coarse river sand. Next occurs loose and nearly clean stratified gravel and boulders, and below this a partially cemented gravel with larger boulders. The material which holds together the gravel and stones of this formation is boulder clay, a porous arenaceous clay silt, through which water passes freely, yet which, in a dry state, will stand in vertical walls to a considerable height. It extends to a greater depth on the higher terraces; in places it is perhaps 500 ft. deep. The boulder clay is here found in two forms: in its original form, as first laid down; and, especially upon the lower benches next to the river, in a secondary or rearranged form. Under the lower benches, particularly under the slips, there is a deposit of silt or imperfect clay, which shows in places to a depth of from 50 to 200 ft. It is the same silt which forms and binds together the boulders of the boulder clay, but is entirely free from gravel or boulders. These lower deposits have been named the white silt deposits. "They are generally fine and uniform in texture, and are usually well bedded in perfectly horizontal layers from ½ in. to 4 ins. in thickness," with occasional sandy seams and small pockets of coarse sand, formed locally, appearing in places.

By the continued application of large quantities of irrigation water upon the cultivated fields above, and upon the upper portions of what are now the slides, almost the entire surplus not absorbed by the plants or evaporated, sank down freely through the loose soil, sand and gravel; and, while not as readily, yet with considerable ease, through the boulder clay, and reached the underlying silt. After some years this water saturated the argillaceous silt and converted it into the form of river mud of about the consistency of thick pea-soup. Long before the whole mass, or even a very large part of it, reached a state of perfect saturation, the silt would lose its power of sustaining weight. In the two places here referred to, on account of the peculiar topographical and geological contour of the country, the water applied at the back was concentrated, comparatively speaking, into one channel of descent (in each place) to the body of silt below, and thence it penetrated in every direction. The process of saturation required many years to produce any results, for if a considerable quantity of the silt had become saturated to the point at which it would lose all cohesion, it would not move, on account of there being so great a distance to any point of outlet, together with the self-supporting power of the boulder clay in its confined position, which was nearly all absolutely dry over the slip; hence a large extent of the underlying silt became more or less saturated before it could find an outlet in any direction, even with a considerable weight upon it in its more or less semi-liquid state. Finally, when a large body of the silt had become saturated to such an extent that it could not sustain even its own weight, except in its confined position, and the limit of resistance, possibly in the form of an arch, of the boulder clay had been reached, the great mass of earth and boulders above—in the case of the south slide, estimated as weighing some 32 000 000 tons, and of the north slide approaching 100 000 000 tons—the whole mass dropped almost vertically, while the immense tracts of broken and mixed material seeking an outlet forced their lower sides out on the line of least resistance and found their way into the river. This action is distinctly shown by the almost vertical walls in the boulder clay along the outline of these two slides. While at their foot there is now a talus slope of crumbled material, these walls stand vertically to a height of from 50 to 200 ft., more clearly shown in the north slide, where the vertical cliffs of boulder clay, and in places of the silt itself, extend around the whole slide for a distance of more than 1.5 miles. It is also shown by the present position of large sections of the original surface of the highest bench, which broke off at the line of the back wall, and which now stand in the sunken mass at an angle of about 45°, with their former level surfaces tilted back and away from the river. The back edge thus dropped first and lower than the portion some distance in front of it. In dropping and pushing out toward the river, the whole tract was broken into sections by great cracks, which still exist. The larger cracks run parallel with the river and at right angles to the line of movement, while other and smaller cracks run in every direction, cutting the whole into blocks of boulder clay and dry silt. Mr. Stanton.

In every instance noted, these slides occurred from 3 to 6 years after irrigation began at each point. In the case of the larger one, the great north slide, the final catastrophe was hastened by the bursting of a small reservoir. A very large quantity of water was necessary for raising crops, on account of the sandy nature of the soil and the nature of the subsoil. The topography of the several benches assisted materially toward the final result. Each field being in the form of a shallow basin, around which the irrigation ditches were built, little of the surplus water was drained off; hence the greater part of that not taken up by the plants or by evaporation ran toward the center of the field and soaked down in one channel.

A most important question here arose, and was pressed hard in the trial of a suit for injunction before the Supreme Court of British

Mr. Stanton. Columbia, which suit went through all the courts of appeal of Canada and was finally settled in favor of the Railroad Company in the Privy Council in London. The question was this: How could the silt, which melted so quickly into slimy mud in still water, stand in vertical walls 100 ft. high, and resist the action, for centuries, of the river running against its sides and not melt down at all?

As to the action of the water upon the peculiar masses of silt which at present underlie the benches and terraces along the Thompson River, a number of curious facts were noted in and around the south slide, which at first seemed most difficult to explain. The silt or imperfect clay, which lies at some points in this section in masses from 200 to 1 000 ft. in thickness, is generally fine and of uniform texture, and is usually well bedded in horizontal layers of from ½ in. to 3 or 4 ins. thick. In its natural state it is hard and dry, like a soft sandstone, and, when held between the fingers and struck with a light hammer, rings like stone. A large piece of this silt, however, placed in a basin of water dissolves after a few minutes and falls down, not in a lump as clay, but mingles with the water, forming a semi-fluid mass like thick pea-soup. The same soft mixture was observed oozing out at many points along the foot of the slide, forced out by pressure from above; so the question arose, how was it that this silt stood in vertical walls from 10 to 100 ft. in height along the Thompson River, with the waters of the stream running along and against their base, and at high water some distance up them, and yet they had stood for ages, and were but little injured, except by slight atmospheric disintegration?

The silt is formed of three principal parts—silica, in the form of coarse and fine sand, and alumina in two forms, first, disintegrated feldspar, simply separated mechanically into grains, both coarse and fine, also in the form of sand, and constituting a large part of the mass; and second, decomposed feldspar or plastic clay. Under the action of running water, the sand, both the silica and the disintegrated feldspar, is washed out of and off from the surface, while the decomposed feldspar is precipitated and forms a coating of true plastic clay upon the mass, which soon becomes impervious to the water and is practically indestructible, thus protecting the underlying silt from further action of the water. The result of a chemical examination * of the material is given in the paper referred to.

Mr. H. J. Warsap, the chemist who made the analysis, also suggests the chemical action and assistance of the carbonic acid of the atmosphere, and ammonia, if present in the clay and the lime, in forming this impervious coating. The mechanical action of separation and precipitation noted above, it is believed, accounts for all the peculiar phenomena observed at every point in and around the slides.

* *Minutes of Proceedings*, Inst. C. E., Vol. CXXXII, p. 20.

The great quantity of irrigation water soaked downward into the mass of silt. It would absorb 53% of water without changing its form, yet with only about 35% it would be incapable of sustaining any great weight except in its confined position. After the final breakdown, and its release into the river with the continual application of water, and still being under pressure, this semi-liquid silt, containing all its original constituents, is forced out at the foot of the slide in great quantities. If a man steps on this ooze he is likely to sink into it, while, within only a few feet of such a spot, when examined by the speaker, there lay a large block of the same silt which had fallen over into the river in a dry state, and over which the last season's high water had run; it stood up 3 or 4 ft. above the level of the river beach, and the speaker walked and jumped upon it without making any impression. Breaking off a piece of this block, it was found that, less than 1 in. under the surface, the silt was in its original form, and easily dissolved in water. In the river, under low water, were also observed great masses of this silt which had fallen over into the river in blocks, over the surface of which the river had run for years without carrying them away. On the other hand, the backwater in an eddy soaking through the cracks and getting behind and around other blocks dissolved them completely, and the river carried them away. Mr. Stanton.

It has been suggested by Emil Kuichling, M. Am. Soc. C. E., that there may be other causes which enter into the formation of the protecting surface so quickly placed on the mass of silt or clay when exposed to the action of running water and which thus prevents its destruction. The particular additional science to which Mr. Kuichling refers being that of biology. Mr. Kuichling has made quite extensive study and examination into the development of the vegetable, and by some thought to be even animal, life on the surface of newly exposed clay banks, especially in brick yards.

It is not for the speaker to refute any such scientific suggestion when coming from Mr. Kuichling. The subject is one in which the speaker has been somewhat interested, and has observed and noted the growth of vegetable life, and its undoubted protection, upon almost vertical railroad cuts, along the Missouri River between Omaha and Platsmouth, Nebraska, where from his personal knowledge, these banks have stood for more than twenty years, still showing the marks of the picks used in their first excavation, and also upon cuts on railroads and common wagon roads, in the neighborhood of Vicksburg, Mississippi.

The part that biology may play in the protection of all these banks or walls of clay or silt is a most interesting one, and one that should receive careful study, but it is not one that the speaker can now enter fully into for two reasons: First, not having the necessary accurate and detailed information, and secondly, not having the time to give this subject sufficient study.

Mr. Stanton. Still there are one or two considerations which would seem, without further knowledge, to answer partly, if not completely, Mr. Kuichling's suggestion when applied to the vertical walls of silt on the Thompson River.

The action of the running water, in taking away the silica and feldspar sand, and precipitating the plastic clay protection, is almost instantaneous; if it were not so, the wall would crumble and be washed away.

Is it possible that plant life, even though the seeds, howsoever minute and howsoever numerous, exist, as stated, in the clay itself, could germinate, take root, and form such protection in an inappreciable instant of time?

That such plant life does form and protect many clay banks, as observed and noted on the railway cuts in Nebraska and Mississippi, is entirely correct; but has not such formation required considerable time, as compared with the instantaneous action of the running water on the silt in British Columbia?

Further, from many experiments in the propagation of disease —the very subject which Mr. Kuichling was investigating for the New York Board of Health—it is claimed by very high authority that the germs from which the vegetable and even animal life come are more likely those deposited upon the clay from the air. Not attempting at this time to prove or disprove this claim; if it is true, it would seem that it would require a still longer time to produce such life and growth as would protect the silt walls from the water action.

In either case, it would appear that the mechanical action of the running water, as described above, perhaps coupled with the chemical action, suggested by the chemist, Mr. Warsap, entirely accounts for the observed phenomena on the Thompson River.

However, the suggestion of geology, as a third science to be studied, places more emphasis upon the subject of this discussion— the absolute necessity for engineers engaged in the construction of permanent works, more particularly reservoirs, railway embankments, retaining walls and other structures intended to resist lateral earth pressure, to study, besides mathematics and mechanics, all kindred sciences which may possibly enter into and affect the final conclusions and results.

Hence, it is particularly urged upon young engineers, and in fact upon all, before constructing such works, to find out first exactly what the material really is, both geologically, chemically and otherwise, and then calculate mathematically its mechanical operation.

Perhaps it may not be out of place here to ask a question or two which, it is hoped, may bring out discussion which will throw light upon this general subject.

Has not the modern specializing of the engineering profession into the numerous special branches tended to cause many, particularly young, engineers, to feel, and, to their injury, to act upon the feeling, that their particular branch was the one all important, and thus be led into such errors as noted in connection with all previous investigations of the British Columbia land slides? Mr. Stanton.

No one appreciates more fully than the speaker the advantages of special technical education and a thorough knowledge of one subject at least, and the disadvantages of its absence, and thus possibly being a "Jack of all trades," but did not the old-fashioned practice of civil engineering—which embraced everything except military works—impel the engineer, placed far out on the confines of civilization, and thrown upon his own resources, to look upon the problem in hand from more different sides, and not only through his own particular hobby, so to speak, and thus bring him nearer the composite truth, than the modern specialist, under the same circumstances?

These considerations are not advanced as dogmas or even positive opinions, but for the purpose and in the hope of drawing out from engineers, more competent to speak, their opinions on one side or the other.

Richard Lamb, M. Am. Soc. C. E.—This subject is of great importance. Mr. Stanton seems to infer that the paper is more theoretical than practical. The speaker differs from him. Mr. Lamb.

Those whose business is the designing of material-handling plants are aware of the paucity of empirical data in regard to lateral pressures of various materials, and of practical conclusions deduced from such experiments. It would have been of benefit if Mr. Goodrich could have extended his experiments to a number of materials, such as coal, ashes and mineral ores, and given his deductions therefrom.

In designing coal bins for use in New York City, the Building and Dock Departments require that in calculating the lateral stresses, 45° be used for the angle of sliding of bituminous, and 30° for anthracite coal. Tests made by the speaker indicate that dry bituminous coal will slide down an iron chute placed at 40° to the horizon, and down a wooden chute at 45 degrees. Anthracite will slide down iron at 30° and down wood at 35 degrees. In freezing weather, iron chutes often become inoperative at the angles above named.

In bins with flat bottoms and with side gates, the problem is changed; and the angle of repose due to friction of coal upon coal has to be considered, instead of coal upon wood or iron. Piles of bituminous coal, especially when lumpy, will stand almost vertically. In determining the lateral stress, as the vertical component is due to the weight of the coal contained between the vertical wall of the bin and the angle of repose of the coal, the lateral stress is inversely proportional to the angle of repose. If the Building and Dock Departments require the use of a lesser angle than the actual conditions demand, de-

Mr. Lamb. signers are compelled to provide a much more expensive structure than the so-called "practical builder" constructs successfully in other cities. The foregoing facts, together with the rules prescribed in the new Building Laws now in effect in New York City, impose a heavy and unnecessary burden upon the coal trade. A series of tests with coal, like those made by Mr. Goodrich, would permit of the establishment of standard data, which would justify the Building and Dock Department engineers in lessening their requirements.

The following formulas are used, by one of the leading coal-handling machinery firms, in calculating the tabulated statement, as shown in their catalogue, giving horizontal pressures of coal at various elevations, upon the sides of coal bins. These formulas were not deduced from experiment, but are based entirely on theoretical grounds. If there are any errors, they are on the side of safety, as many iron and wooden coal bins have been built, the calculations being made with these formulas. If experiments were made, undoubtedly the angles of slip would be increased and the bins made by the new formulas would be less expensive. With a coefficient of safety of 5, the extra strains imposed by frozen coal dropping, or an avalanche of coal, would be amply safeguarded.

PRESSURE EXERTED BY COAL AGAINST VERTICAL RETAINING WALLS, PER FOOT OF LENGTH.

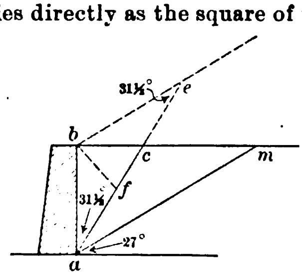
Angle of repose of hard coal = 27 degrees.
Weight of coal per cubic foot = 52.1 lb.

For Surface Horizontal.—

$$\text{Vertical pressure} = \frac{\text{weight of the triangle, } a\,c\,b \times b\,c}{a\,b}$$

$$= \frac{52.1 \times a\,b \times b\,c \times b\,c}{2\,a\,b} = 26.05\ (b\,c)^2,$$

or, the pressure varies directly as the square of the depth, d.



When the Surface Slope is Equal to the Angle of Repose.—

$$\text{Vertical pressure} = \frac{\text{weight of the triangle, } a\,e\,b \times b\,c}{a\,b}$$

$$= \frac{52.1 \times b\,f \times a\,f \times b\,e}{a\,b} = 52.1\ (b\,f)^2,$$

again, the pressure varies directly as the square of the depth, d.

Hard Coal.— Mr. Lamb.

Surface horizontal $\begin{cases} \text{Total pressure for depth, } d = 9.78\ d^2. \\ \text{Pressure on lowest foot } = 9.78\,(2d-1). \end{cases}$

Surface sloping $\begin{cases} \text{Total pressure for depth, } d = 14.22\ d^2. \\ \text{Pressure on lowest foot } = 14.22\ (2\ d-1). \end{cases}$

Soft Coal.—

Surface horizontal $\begin{cases} \text{Total pressure } = 6.37\ d^2. \\ \text{Pressure on lowest foot } = 6.37\ (d-1). \end{cases}$

Surface sloping $\begin{cases} \text{Total pressure } = 10\ d^2. \\ \text{Pressure on lowest foot } = 10\ (2\ d-1). \end{cases}$

Angle of repose = 35 degrees.

The speaker was recently engaged in designing a plant for handling the ashes and street sweepings of Brooklyn by trolley cars, and could find no data giving the angle of repose of ashes and street sweepings. He decided, by tests, that with iron chutes the angle should be not less than 30°, and designed a car with 34° for the angle of the floors. This car was not built, because it was learned that Col. Waring had a number of scows built with an angle of about 32° for the floors. They were expected to be self-discharging, but proved to be failures. Subsequently, the writer constructed some buckets, of about 7½ tons capacity, having trunnions and being designed to be dumped by derrick car and cableway. These buckets can be turned upside down. From operating these buckets, it has been observed that they will discharge the ashes when at an angle of about 35 degrees. On the strength of this information, the writer is now designing a car with mechanism for discharging these large buckets from the cars direct.

Ashes are often seen piled almost vertically. The lateral pressure of ashes, especially when bound together with rags, and other matter in the street sweepings, cannot be determined by any data available in the books of our profession, as far as the speaker has been able to learn.

Mr. Goodrich determined from his experiments the fact that the more finely a material is divided, the less the lateral pressure. He stated, however, that he considered that this deduction is of more theoretical than practical importance. On the contrary, the speaker wishes to emphasize the practical importance of this very deduction. Had this matter been brought to the speaker's attention before designing and building a large copper mining plant recently, he would have designed the ore bins differently. Steel floors, at an angle of 40°, were put in, and it was found that the crushed quartz ore, from 1½ in. to ⅛ in. in size, would slide down readily, but the very fine ore, up to ⅛ in. in size, would not move. A water jet, extending across the bin, had to be supplied in order to make the quartz fines move down a 40° steel-incline floor. It is evident, therefore, that the finer the ore is crushed, the less will be its lateral pressure when confined in a bin, which is what Mr. Goodrich proved by his experiments.

Vol. XXX. AUGUST, 1904. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

LAKE CHEESMAN DAM AND RESERVOIR.

Discussion.*

By Messrs. Edward Wegmann, J. Waldo Smith, E. Kuichling, R. Shirreffs, George Y. Wisner and Edwin Duryea.

Mr. Wegmann. Edward Wegmann, M. Am. Soc. C. E.—The paper shows that the Cheesman Dam has been well designed and built. It will be the highest structure of its kind until the New Croton Dam shall have been completed. The profile of the dam was determined on the principle that the dam was to resist the water pressure by its weight alone, and the curving of the plan of the wall appears to have been done principally to obtain an economical location, as is shown clearly on Plate XVII. That this arching adds materially to the strength of the structure to resist overturning, will be admitted readily by most engineers.

Two dams have been constructed, which owe their stability solely to the fact that they are curved in plan, so as to act as horizontal arches, *viz.*, the Zola Dam, in France, and the Bear Valley Dam, in California. The first of these dams was built in 1843 for supplying the City of Aix with water. It has a maximum height of about 120 ft. and a length on top of only 205 ft. At the top and bottom the dam is, respectively, 19 and 42 ft. wide. The dam is curved in plan, the radius at the crown being 158 ft. When the reservoir is full, the line

*Continued from May, 1904, *Proceedings*. See March, 1904, *Proceedings*, for paper on this subject by Charles L. Harrison, M. Am. Soc. C. E., and Silas H. Woodard, Assoc. M. Am. Soc. C. E.

of pressure at the base falls 11.48 ft. outside of the structure, which, therefore, would be overturned, if it were not curved in plan. Mr. Wegmann.

The Bear Valley Dam, built in 1884, has a maximum height of 64 ft. It is 3.17 ft. wide at the top and 20 ft. wide at the base. The plan of the dam is curved, with a radius of 335 ft. When the reservoir is full, the line of pressure at the base falls about 15 ft. outside of the dam.

It is a simple matter to determine the profile of a dam which is to resist either solely by gravity or only as an arch. However, when both actions are assumed as occurring at the same time, and an endeavor is made to calculate the relative resistance offered by each, the problem becomes quite complicated. The French engineers, Delocre and Pelletreau, gave solutions* of this problem, more or less satisfactory. To these must now be added the solution by Mr. Woodard, given in the latter part of the paper.

In applying mathematics to this problem, we are obliged to assume, in our present state of knowledge, that the masonry of the dam is perfectly elastic, which causes the strains to be according to a "uniformly varying stress." That this assumption, when applied to a masonry dam 150 ft. thick at the base, may be far from the truth is recognized fully by Mr. Woodard. While complimenting him on the mathematical skill he has shown in dealing with this intricate problem, the speaker questions whether any reliance can be placed on the results obtained by formulas based on such uncertain assumptions. Professor Rankine, in his "Report on the Design and Construction of Masonry Dams,"† reached the conclusion that there was such uncertainty in trying to calculate the resistance offered by a curved dam, due to its weight and the arch action at the same time, that he recommended that a dam be always made sufficiently strong to resist the water pressure by gravity alone; and that, in case of a narrow valley, the plan be curved so as to give some additional, though unknown, strength to the structure.

This is exactly what was done in the case of the Cheesman Dam, and Mr. Woodard's formulas appear to have been an after-consideration to determine just how much resistance was offered by the weight of the dam and how much by the arching, both being assumed to act at the same time. If these formulas, however, are to have their full practical value, they must be considered sufficiently reliable to enable us actually to reduce the profile of a "gravity dam," when curved in plan. Probably few, if any, engineers would be willing to apply Mr. Woodard's formulas to that extent.

Mr. Woodard states that the usually accepted method of determining the profile of a dam resisting the water pressure by gravity

* *Annales des Ponts et Chaussées* for 1866, 1876 and 1877.

† See *The Engineer*, January 5th, 1872.

Mr. Wegmann. alone is simple and, with variations in details, may be found in text-books and works on masonry. Twenty years ago, when the calculations for the New Croton Dam were made, this was by no means the case. The only methods that could be found were those of the French engineers, Sazilly, Delocre and Pelletreau, involving equations of the fourth and sixth degree, and that of Professor Rankine, for a profile bounded by logarithmic curves. All these engineers designed "profiles of equal resistance," *i. e.*, profiles in which the maximum pressures at the faces of the dam were kept at a uniform limit. From this resulted profiles with curved faces.

In designing the New Croton Dam, the principle followed was to limit the lines of pressure, reservoir full or empty, to the center third of the profile, allowing the pressures at the faces to increase gradually until they reached the limit of safety, which had been assumed. Below this point the slopes of the faces were increased so as to keep the maximum pressures at the limit of safety. By simple equations of the second degree, the thickness of the dam was calculated at intervals of 10 ft. This gave a profile with polygonal faces, having many changes of batter, for which, in the final design, curves, or a few straight lines, were substituted.

This method, which was first published in 1887, in the Report on the Quaker Bridge Dam, by the late A. Fteley, Past-President, Am. Soc. C. E., Consulting Engineer of the Aqueduct Commissioners of New York, and in the following year, by the speaker, in a book on dams,* appears to have been adopted in the design of most high American dams since that time. From some of the details given in the paper and from the general look of the profile, the speaker concludes that the Cheesman Dam was designed by the method just mentioned.

Mr. Harrison has described the high rock-fill dam which was begun at the site of the Cheesman Dam and was destroyed by a flood. Considering the large cross-section which had to be given to the rock-fill, it is evident that such a structure could only be justified when the cost of cement delivered on the work was very high, and it would be interesting to know what the difference in cost would have been between the rock-fill and the masonry dam.

The details of the outlet tunnels, valves, etc., show skilful engineering.

The authors are to be thanked for their full description of this important high dam, and for giving the profession Mr. Woodard's method of treating a curved dam simultaneously as a horizontal arch and a "gravity section."

Mr. Smith. J. WALDO SMITH, M. AM. SOC. C. E.—Mr. Harrison has discussed this paper so thoroughly, from the speaker's standpoint, that little

* "The Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E.

more can be said. It would be interesting, however, to hear an answer to Mr. Kuichling's question regarding the temperature strains or the cracks from internal stresses in the dam. This is a subject which is of great interest to engineers at the present time, as several are studying what is happening in dams which have been built during the last eight or ten years in reference to various cracks which have developed from one cause or another. Mr. Smith.

Measures have been taken to ascertain the temperatures in the Boonton Dam by introducing a number of thermophones, from which the temperatures are to be read during a long period. These were put in during last season, but at the present time sufficient data have not been obtained from which to draw conclusions. It can only be said in a general way that the temperature in the interior of the dam is still falling, and has been falling steadily from the start. The second point shown is that no matter what the temperature of the air at the time the masonry is laid, the ultimate maximum temperature of the masonry is approximately the same. The thermophones near the outside of the wall indicate a steady fall through the cold weather, and, as soon as the temperature changes in the spring, they indicate a rise. The daily changes in temperature seem to have considerable influence on the average temperature of the dam, and, although the average monthly temperature is falling steadily, the average daily temperature shows a rise and fall following somewhat closely the changes in the air.

E. Kuichling, M. Am. Soc. C. E.—This is an extremely interesting and valuable paper, as it presents clearly the essential details of the design and construction of a great masonry dam. In this respect, the author's description leaves little to be desired; but, inasmuch as the hydrology of the watershed under consideration is peculiar, and differs greatly from that of drainage basins of similar magnitude in other parts of the United States, it has appeared to the speaker that somewhat more space might have been given advantageously to the rainfall and run-off of the South Platte River Basin. Mr. Kuichling.

The author states that the maximum flood flow of the river at the site of the dam is only 1 945 cu. ft. per second, from an area of 1 796 sq. miles. This is at the rate of but 1.08 cu. ft. per second per square mile, which is remarkably low when the rainfall statistics for Denver are considered. From the latter are found the following monthly maxima, in inches, during 32 years (1872–1903):

January...... 2.35	May....... 8.57	September.... 3.70
February.... 1.22	June....... 4.96	October.. ... 3.92
March....... 3.10	July....... 4.28	November.... 1.93
April........ 8.24	August.... 2.84	December.... 2.32

and from the much greater altitude of the water-shed, the average elevation of which is about 8 000 ft. above the sea, it may be presumed that the precipitation is somewhat more than at Denver.

Mr. Kuichling. It is also found, from the hydrographical records of the United States Geological Survey in this region, that the river freshets usually occur in May and June; and, as the run-off in these months may readily amount to 75% of the rainfall, it follows that from the precipitation of 8.57 ins. in May, 1876, and 4.96 ins. in June, 1882, run-offs of, respectively, 6.43 and 3.72 ins. in depth on the water-shed might have occurred. Now, as a depth of 1.0 in. run-off in 30 days corresponds to an average discharge of about 0.9 cu. ft. per second per square mile, and as the maximum rate of discharge during the month may easily be twice the average, it also follows that, in the two months mentioned, freshets of about 12 and 7 cu. ft. per second per square mile of water-shed might have been expected.

While these figures of run-off are merely speculative, they are much less than has been observed from similar monthly rainfalls in the streams which drain the mountainous regions of the Eastern States, and hence a more extended description of the meteorology and physical characteristics of the drainage basin of the South Platte River will add much to the value of the paper. It may also be noted that a run-off equal to a depth of 0.84 in. on the water-shed will fill the reservoir, and that such a quantity may be expected from the above-mentioned maximum precipitations for November, December and January.

Another matter of interest is the author's assumption that during the winter months the reservoir will never be full, but will always be at a low stage, so that any thrust on the dam due to the expansion of the ice on the surface of the water may be neglected. A further reason for not considering ice thrust is the circumstance that a rocky promontory a few hundred feet above the dam will serve as a protection against the push of the ice, even though the reservoir may be filled to near the spillway level.

From the meteorological data, however, it seems to the speaker very possible that the reservoir may become full during freezing weather, and may remain so for a few weeks if the outlet valves are not regulated carefully or if they become disabled. In this event, ice will form at a high level, and, by its expansion when warmed by the heat of the sun during the daytime, a considerable thrust will be exerted against the dam.

As to the magnitude of the thrust which is actually exerted by the ice sheet on a reservoir, few data seem to be available. Some writers have considered that it is limited only by the crushing strength of the ice, which is from 200 to 1 000 lbs. per square inch, or from 14.4 to 72.0 tons per square foot, depending on its quality and temperature; while others have taken the elasticity of the ice into account, assuming a modulus, E, varying from about 180 000 to 360 000 lbs. per square inch, and a rate of expansion varying from about $\frac{1}{1500}$ to $\frac{1}{4000}$, in passing from 0° to 32° Fahrenheit.

Applying these latter figures to the case of a sheet of ice, 500 ft. wide between the dam and the rocky promontory mentioned, it will be found that the elongation will range from $\frac{1}{3}$ to $\frac{1}{8}$ ft., or from 4.0 to 1.5 ins.; and if $E = 180\,000$, the compressive stress in the ice will be from 120 to 45 lbs. per square inch, while, if $E = 360\,000$, the stress will be from 240 to 90 lbs. per square inch. On this basis of computation, the probable stress ranges from 45 to 240 lbs. per square inch, or from 3.24 to 17.28 tons per square foot; and if the ice is assumed to be 1 ft. thick, the thrust against the dam, accordingly, will be from 3 to 17 tons per linear foot. Mr. Kuichling.

It may also be remarked that in the consideration of this question the Board of Experts on the projected Quaker Bridge Dam, in the valley of the Croton River, New York, recommended in 1888 that provision should be made for an ice pressure of 21.5 tons per linear foot at the spillway level; and that, in designing the large dam which is soon to be built across the Scioto River near Columbus, Ohio, Samuel M. Gray, M. Am. Soc. C. E., adopted an average ice thrust of 11.33 tons per linear foot.

In view of the widely varying data relating to the crushing strength, elasticity and rate of expansion of ice, the speaker endeavored, a number of years ago, to deduce an approximate measure of the thrust of an extensive field of ice by computing the shearing resistance of the masonry piers of several highway bridges which were a very short distance above certain river dams. The water above these dams usually had only a slight mean velocity in winter, but the discharge was then generally sufficient to produce a moderate depth of overflow, thus allowing the ice below the bridge to expand or contract freely, while the long field above the bridge was held by the piers and banks.

In all these cases the piers had remained undisturbed for many years, although the ice was seldom less than 1 ft. thick in midwinter. The weakest piers of the series were 4.5 ft. thick, 18 ft. long, 11 ft. high above the usual level of the ice, and 68 ft. apart, carrying a light iron structure the weight of which did not exceed 550 lbs. per linear foot; hence the total weight resting on the course of masonry immediately below the ice was about 185 000 lbs. The mortar was not strong, and had been partly washed out of the joints, so that a high value for its shearing strength could not be taken. Allowing 5 lbs. per square inch for such shear and 0.6 for the coefficient of friction, the total resistance of the pier to sliding becomes about 169 000 lbs., or 37 600 lbs. per foot of its thickness; hence, in this case, the ice thrust was certainly less than 18.8 tons per linear foot; and, if the total resistance be computed with 0.7 as coefficient of friction including shear, the thrust will be less than 14.4 tons per linear foot.

All these computations for the ice thrust on dams with vertical

Mr. Kuichling. backs are based on data so crude that the appearance of a better array of observations becomes very desirable; and, in view of the importance of the subject, it is earnestly hoped that those who have had the opportunity to measure this force of Nature will contribute to our stock of knowledge concerning it.

Another matter of interest is the behavior of the masonry of the Cheesman Dam with regard to changes of temperature. The speaker was informed recently that during the past winter several vertical cracks made their appearance in the upper portion of the structure. If this be true, it follows that the method of computing a curved dam by considering it as an elastic and continuous arch must be modified.

Mr. Shirreffs. R. SHIRREFFS, M. AM. SOC. C. E. (by letter).—The writer has been greatly interested in the second part of this paper, relating to the stresses developed in a curved dam, not only because he has at present in charge a project where a curved dam is as emphatically "indicated" as it was at the Lake Cheesman site, but also because it fell to his lot a few years ago to direct, under F. P. Stearns, M. Am. Soc. C. E., a study of the merits and demerits of the curved *vs.* the straight form for the Wachusett Dam, then proposed, and now being built by the Metropolitan Water Board, of Massachusetts.

The studies were made in a similar way to those now presented by Mr. Woodard, but with fewer assumptions not borne out by the facts. Unfortunately, the writer's memorandum of these calculations was destroyed by fire some time ago, and what follows has been worked out entirely anew and, it is believed, with some improvement on the methods then followed.

Upon examining the methods followed in the paper, it will be found that several assumptions or approximations are made which do not conform strictly to the actual conditions. The writer believes that some of these can be eliminated without increasing very much the labor of calculation, and with a marked gain to the advantages shown by the curved dam. In the case of the Lake Cheesman project it appears that the only purpose of the analysis was to justify the use of a "gravity" profile when built as a curved dam. The errors occasioned by the assumptions made, as these were nearly all adverse to the vertical beams, were on the side of safety, but they are misleading when applied to the general question of the curved *vs.* the straight dam. And here it may be proper to emphasize the point, to which attention has already been called in the discussion of this paper, that the straight dam is only a special case of the curved dam, and that it is not possible, simply by building a dam straight, to avoid the stresses caused by the combination of vertical with horizontal beams. The real point is whether it is better to build the dam straight, with the certainty that there will be tensile stresses in the horizontal beams, or to avoid all tensile stresses by building the dam on a curve when the site will permit the use of a short enough radius.

Mr. Shirreffs.

The approximations referred to are these:

1.—In ascertaining the deflections of the vertical beams, an average value of the moment of inertia was used for each different vertical section;

2.—The deflections of the arch rings were computed as if due to the constant radius of the extrados of the arches; and, most important;

3.—The arch deflections were computed (*a*) as if there was a hinge point at the abutments, and (*b*) as if, even then, there were no moment stresses in the arch.

As an illustration of the errors resulting from this last assumption, and also as an illustration of the advantage of reducing the radius when it can be done, the results of the computation of two curved dams of triangular section, No. 1, having a radius of 400 ft. and a face batter of 0.5477 ($b^2 = 0.30$); and No. 2, having a radius of 260 ft. and a face batter of 0.40, are given in Table No. 6, the first set of figures resulting from the arch deflections obtained by Mr. Woodard's method and the second set resulting from deflections obtained by treating the arches as curved beams. In both sets the approximations noted above under "1" and "2" have been corrected. Both profiles are supposed to be built in a triangular valley similar to that sketched in Figs. 18 and 19, that is, with a maximum depth of water of 180 ft. and a chord width of 500 ft., at a point 30 ft. below the water line, and with the contours opening outward, up stream, so as to be found in the same relative position on the arcs of the curved dams as on the straight line across the valley. The whole height has been divided into five arch slices of 30 ft. each, as in the example in the paper.

TABLE No. 6.

Joint.		1	2	3	4	5	6
	Q	63 280	112 500	168 750	225 000	281 250	....
Profile No. 1........	P	43 600 37 650	97 280 85 980	156 060 147 280	206 780 211 070	278 550 275 680	....
	$\frac{u+v}{l}$	....	0.602 0.561	0.679 0.628	0.724 0.672	0.754 0.706	0.782 0.736
Profile No. 2........	P	15 320 11 520	59 620 41 580	122 360 86 100	197 750 172 680	271 700 261 840	....
	$\frac{u+v}{l}$	....	0.469 0.434	0.644 0.554	0.762 0.684	0.867 0.733	0.960 0.836

In Table No. 6, Q is the total water pressure, in pounds, considered as concentrated at the various joints; P is the part of this load carried by the vertical beam at the center of the dam, and $\frac{u+v}{l}$ the

Mr. Shirreffs. ratio of the distance of the resultant pressure from the up-stream edge of the joint to the length of the joint. (See also the arch deflections in Table No. 9.)

It will be seen that the use of the true arch deflections has a considerably greater effect in No. 2 than in No. 1 in reducing the eccentricity of the resultant pressure beyond the "middle third" at the lower joints. It is entirely probable that the necessary modification of either profile to give it a reasonable top thickness will fully correct this eccentricity in either. At this point attention may be called to the superior ability of a curved dam to resist shocks, as of ice or drift,

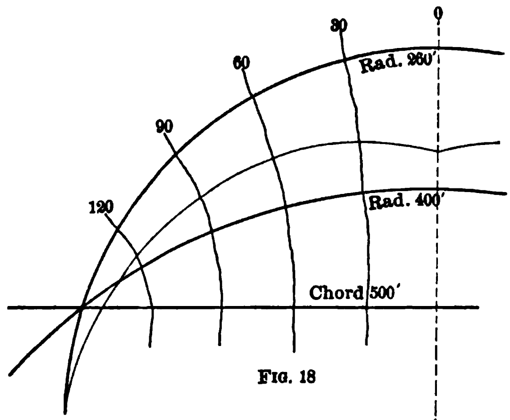


FIG. 18

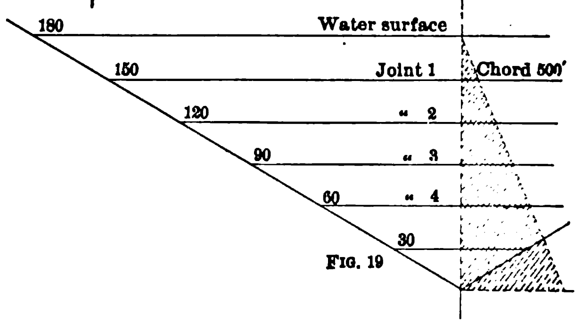


FIG. 19

for which a top thickness is necessary; especially in respect to the most dangerous of these effects in a cold climate—the thrust of a field of ice expanding on a bright, sunny day succeeding an extremely cold night. It is probable that this superior ability will justify the use of no more masonry in the top of a curved dam than would be used in a straight dam of the same height, or that, in other words, the top thickness having been determined for a straight dam, the curved dam may be given a lesser thickness in the direct proportion of the chord to the arc.

The volume of Profile No. 1, if built in the assumed valley, will be 67 200, and that of No. 2, 58 490, cu. yd., a difference of 8 310 cu. yd. in favor of the latter, or about 12 per cent. A gravity profile would have a face batter, $b = \sqrt{\frac{1}{\delta}}$; δ being the ratio of the weight of masonry to that of water. Assuming this, with Mr. Woodard, and as has been done in the above computations, to be 2.5, the batter of the gravity profile would be 0.63246. A straight dam across the assumed valley with this section would contain 63 250 cu. yd., while the same section built on radii of 400 and 260 ft. in the same valley would contain, respectively, 77 120 and 91 880 cu. yd. Mr. Shirreffs.

The economy of the curved dam, however, must obviously be determined specially for each situation. At the Lake Cheesman site, for instance, a straight dam was evidently out of the question. It appears from Mr. Maltby's discussion* that a radius of 300 ft. was at one time proposed for this dam, and from the contour plan (Fig. 1, Plate XVII),* it would seem that a radius as short as 260 ft. would have increased the top length only about 60 ft. and the lengths below Elevation 150 (water surface at Elevation 214) only about 15 ft. as an average. The economy of the curved dam on 400 ft. radius, therefore, would have been about 13%, as compared with a "gravity" section on the same radius; while the curved dam on 260 ft. radius would have saved, say, 36% over the gravity dam on the same radius and about 27% over the curved dam on the longer radius.

In the foregoing computations the dam with triangular profile was chosen because all the mathematics are simpler for this form. It is also the theoretical form for a gravity dam as well as for an arched dam, if, in the latter case, it be considered that the arch elements carry all the load, and if it be further assumed (which is not very far from the truth) that the average normal stress at all points is $S = \frac{q\,R}{A}$. In making the first studies for any new situation it will probably save time to use the triangular profile for the first approximations, afterward making the necessary modifications to give the top a reasonable width. The caution may here be noted that instead of adding to the triangular form a parallel top there should be added only a moderate top width, with a change of the batter for a considerable distance. If a considerable top width be added to the triangular profile the result will be negative loads on the vertical beams toward the top, giving tension at the upper joints at the down-stream face.

Passing now to a detailed description of the modifications proposed in the methods used by Mr. Woodard, these will be taken up in the order in which they are noted above.

* *Proceedings*, Am. Soc. C. E., for May, 1904.

Mr. Shirreffs.

First.—The following method is proposed to obtain the deflections, under the action of a series of concentrated loads, of a beam having a varying moment of inertia, but a constant thickness = 1.0. It is necessary that the beam be limited by straight lines between the load points. Using the notation shown in Fig. 20, and, in addition, the following:

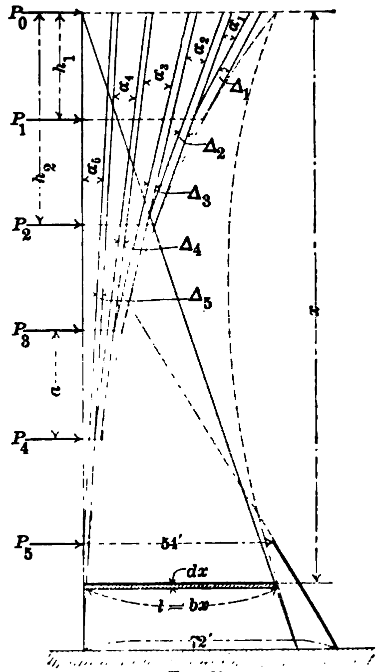


FIG. 20

b = ratio of increase of joint length;

I = moment of inertia of joint;

e = distance from neutral axis to extremity of joint;

M = moment of all loads above joint considered;

σ = stress at extremity of joint;

λ = change of length of element, dx, of beam, due to σ;

E = modulus of elasticity of material (in compression);

ρ = radius of curvature of deflected beam;

α = angle of deflected beam at any load point, having reference only to the portion of the beam between that load and the one next below;

Δ = deflection of each individual portion of beam, produced by all loads above that portion;

D_n = the total deflection of beam at any load point:

We have, generally:

$$\sigma = \frac{M e}{I}; \ \lambda = \frac{\sigma \, d x}{E}; \ \rho = \frac{E I}{M}, \text{ and } d \alpha = \frac{d x}{\rho}.$$

But (referring for the present only to the beam of triangular profile),

$$e = \frac{l}{2} = \frac{b x}{2}; \ I = \frac{l^3}{12} = \frac{b^3 x^3}{12}, \text{ and } d \Delta : \lambda = (x - h) : e;$$

therefore,

$$d \Delta = 12 \, M \, (x - h) \frac{d x}{b^3 \, E \, x^3} \dots\dots\dots\dots\dots\dots (1)$$

$$\text{and } d \alpha \text{ (or also } d \tan. \alpha) = \frac{12 \, M \, d x}{b^3 \, E \, x^3} \dots\dots\dots\dots (2)$$

But we can write $h = n\,a$, and therefore, Mr. Shirreffs.

$$M_n = P_1\,(x - a) + P_2\,(x - 2\,a) + \ldots\ldots + P_n\,(x - n\,a).$$

The reduction and integration of the resulting equations being quite simple, these pages will not be cumbered with anything except the final results. After obtaining the values of Δ_n and tan. α_n, these must be cumulated as follows for D_n:

$D_5 = \Delta_5$;

$D_4 = \Delta_5 + \Delta_4 + a$ tan. α_5;

$D_3 = \Delta_5 + \Delta_4 + \Delta_3 + a$ (2 tan. α_5 + tan. α_4);

and so on, as will appear from Fig. 20.

Before assigning values to either a or b, we have :

$$b^3\,E\,\Delta_5 = 12 \sum_5^1 P\,(L\,n\,6\,a - L\,n\,5\,a) - \frac{1}{30}\,(61\,P_1 + 62\,P_2 + 63\,P_3 + 64\,P_4 + 65\,P_5)\,;$$

$$b^3\,E\,\Delta_4 = 12 \sum_4^1 P\,(L\,n\,5\,a - L\,n\,4\,a) - \frac{6}{100}\,(41\,P_1 + 42\,P_2 + 43\,P_3 + 44\,P_4)\,;$$

$$b^3\,E\,\Delta_3 = 12 \sum_3^1 P\,(L\,n\,4\,a - L\,n\,3\,a) - \frac{1}{8}\,(25\,P_1 + 26\,P_2 + 27\,P_3);$$

$$b^3\,E\,\Delta_2 = 12 \sum_2^1 P\,(L\,n\,3\,a - L\,n\,2\,a) - \frac{1}{3}\,(13\,P_1 + 14\,P_2);$$

$b^3\,E\,\Delta_1 = 12\,P_1\,(L\,n\,2\,a - L\,n\,a) - 7.5\,P_2$; ($L\ n\ (a)$ = Naperian Log. (a));

$$a\,b^3\,E\,\text{tan. }\alpha_5 = \frac{1}{150}\,(49\,P_1 + 38\,P_2 + 27\,P_3 + 16\,P_4 + 5\,P_5);$$

$$a\,b^3\,E\,\text{tan. }\alpha_4 = \frac{3}{200}\,(31\,P_1 + 22\,P_2 + 13\,P_3 + 4\,P_4);$$

$$a\,b^3\,E\,\text{tan. }\alpha_3 = \frac{1}{24}\,(17\,P_1 + 10\,P_2 + 3\,P_3);$$

$$a\,b^3\,E\,\text{tan. }\alpha_2 = \frac{1}{6}\,(7\,P_1 + 2\,P_2);$$

$$a\,b^3\,E\,\text{tan. }\alpha_1 = 1.5\,P_1.$$

Assigning to a the value, 30, as in the above and the Cheesman Dam examples, we have after cumulation :

$b^3\,E\,D_1 = 7.3344\,P_1 - 3.8501\,P_2 - 1.8178\,P_3 - 0.6990\,P_4 - 0.1545\,P_5$;

$b^3\,E\,D_2 = 3.8501\,P_1 - 2.5168\,P_2 - 1.3178\,P_3 - 0.5323\,P_4 - 0.1211\,P_5$;

$b^3\,E\,D_3 = 1.8178\,P_1 - 1.3178\,P_2 - 0.8178\,P_3 - 0.3656\,P_4 - 0.0878\,P_5$;

$b^3\,E\,D_4 = 0.6990\,P_1 - 0.5323\,P_2 - 0.3656\,P_3 - 0.1990\,P_4 - 0.0545\,P_5$;

$b^3\,E\,D_5 = 0.1545\,P_1 - 0.1211\,P_2 - 0.0878\,P_3 - 0.0545\,P_4 - 0.0211\,P_5$.

Before the arch equations can be combined with the beam equations it will be necessary, of course, to assign a value to b. In applying the foregoing method to a beam of irregular profile, but which can be bounded by straight lines between the load points, it is only necessary to remember that for each beam section there will now be a different origin for x, and a different value for b. For example, if the bottom

Mr. Shirreffs. section of the beam have a joint length of 54 on the top and 72 on the bottom, with a still $= 30$, the origin of x will be at the second load point (see Fig. 20), and the value of b will be 0.60. There should now be written, $x - h = x - 90$, and

$M_5 = P_1 (x + 30) + P_2 x + P_3 (x - 30) + P_4 (x - 60) + P_5 (x - 90)$, and after the substitution of these values in the general equation (1), and its integration and reduction, there will result,

$E D_5 = 1.3679 P_1 + 1.0717 P_2 + 0.7755 P_3 + 0.4793 P_4 + 0.1831 P_5$; while in the triangular profile No. 2, with $b = 0.40$ we have

$E D_5 = 2.4141 P_1 + 1.8931 P_2 + 1.3720 P_3 + 0.8509 P_4 + 0.3298 P_5$.

It is next necessary to modify the results derived for the beam of constant thickness to suit the case of the beam comprised between radial planes 1 ft. apart at the up-stream edge of the dam. It will be readily seen that the elementary deflections will vary inversely as the moments of inertia of the rectangle having the height, l, and a base of 1.0 and a trapezoid having the same height and a base $= 1.0$, but a top, $c = 1 - \frac{l}{R}$, l being the joint length and R the radius of the up-stream face of the dam. The ratio of these moments of inertia is

$$m = \frac{c^2 + 4c + 1}{3(c + 1)}.$$

To introduce this value of I in the equations for $d\,\Delta$ and d tan. α would result in forms very difficult if not impossible of integration. The obvious approximation is to multiply the individual deflections and deflection angles by the reciprocal of m obtained from the joint length at the middle of the section under consideration, before cumulating the individual results for the deflections, D. A test computation shows that the error of this approximation is less than half of 1% at the base of Dam No. 1. It will diminish, of course, toward the top, while it will be somewhat greater for a lesser radius.

Before leaving this branch of the subject it may be remarked that we have, for the triangular profile, the following simple formulas, which aid rapid computation. Let V be the volume and W the weight of a vertical slice of the dam contained between radial planes 1 ft. apart at the up-stream face, u the distance of its center of gravity from the same face, v the distance from the vertical through the center of gravity to the point of the base where the resultant pressure strikes, M_p the total moment of all loads above the base, $\delta\,\gamma$ the unit weight of the masonry, and for the remainder the notation already used.

Therefore:

$$V = \frac{3R - bh}{6R} b h^2; \quad W = \delta\, y\, V;$$

$$\frac{u}{l} = \frac{2R - bh}{6R - 2bh} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$\frac{v}{l} = \frac{6 M_u}{\delta\, y \left(3 - \frac{bh}{R}\right) b^2 h^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Second.—The error resulting from computing the arch deflections as due to an arch having the radius of the extrados, instead of using the radius of the axis, will be corrected in the general solution of the deflections of the curved beam, which follows. Mr. Shirreffs.

Third.—The following is offered as a solution of the problem of obtaining the movements and stresses in an arch subjected to a uniform normal or radial load. As the resulting equations are very much simpler and easier of application than the equations for the circular arch, under whatever load, to be found in the textbooks, the methods and results are given in some detail.

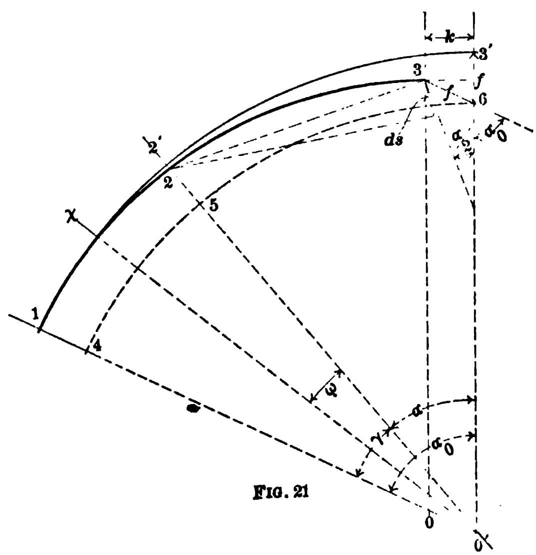


Fig. 21

Imagine the segmental arch, of which 1–2′–3′ (Fig. 21) is one-half in its unloaded position, to be a portion of a "closed ring," or, what is equivalent, imagine the abutments upon which the arch rests to be frictionless, then, under the unit loading, q, producing a shortening, $2\ p$, of the original length, $2\ L$, the radius of the arch will be reduced by a proportionate amount, f, and the half arch will move to the new position, 4–5–6, in which position it will be in perfect equilibrium, with axial stresses at all points $= q\ R$, but without internal moments.

Now imagine the arch to be severed at the crown, the right half being replaced by a crown thrust $= q\ R$, and the half arch then moved outward until Point 4 again coincides with Point 1, but without disturbing the perpendicularity of the axis at the skewback.

Mr. Shirreffs. The center of the arch will now be at 0 instead of 0'; Point 6 will be in the position of Point 3, at the horizontal distance, k, from the vertical through the middle of the span and at the vertical distance, Δ_c, below the original crown of the arch, but the crown joint will still be vertical. The arch will be assumed to have a width, l, and a thickness, 1. Since $f : \lambda = R : L$, and since $\lambda = \frac{q R}{A} \frac{L}{E}$, we will have,

$$k = f \sin. \alpha_0 = \frac{q R^2}{E l} \sin. \alpha_0 \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

and

$$\Delta_c = f (1 - \cos. \alpha_0) = \frac{q R^2}{E l} (1 - \cos. \alpha_0) \ldots\ldots\ldots\ldots (6)$$

At a point distant α degrees from the crown we shall have $\Delta_a : \Delta_c = \alpha_0 - \alpha : \alpha_0$, and, therefore,

$$\Delta_a = \frac{q R^2}{E l} \frac{\alpha_0 - \alpha}{\alpha_0} (1 - \cos. \alpha_0) \ldots\ldots\ldots\ldots\ldots (6a)$$

In order now that the integrity of the arch be restored, it is necessary:

First, that the crown thrust, $q R$, be so diminished that, under the joint action of such diminished thrust and the loads on the half arch, the curved beam, 1-2-3, considered as fixed at the abutment, shall be deflected through the horizontal distance, k; and

Second, that the crown joint, which will have been deflected by this movement through the angle, β, shall be again made vertical in its new position, which can only be accomplished by the application thereto of a moment, M_c. As the original crown thrust, $q R$, just holds the arch in equilibrium against the action of the loads, the first movement will be identical with that caused by the action of a force, H', applied at the crown, equal to the necessary diminution of $q R$, and acting, therefore, toward the right. The total movement of the arch at any point will be obtained by combining the movements produced by H' and M_c with those resulting from the axial stress.

Neglecting the slight reduction in the compression of the arch occasioned by diminishing $q R$ by the amount, H', to obtain the value of H' we must substitute in the general expression for the deflection of a beam, $d s = M x \frac{d x}{E I}$ (moments taken about Point 2), $H' R (1 - \cos. \alpha)$ for M; $2 R \sin. \frac{\alpha}{2}$ for x; $R \, d \, \alpha$ for $d x$ and $\frac{l^3}{12}$ for I, and we get:

$$d s' = \frac{24 R^3 H'}{E l^3} (1 - \cos. \alpha) \sin. \frac{\alpha}{2} d \alpha.$$

But the horizontal component of this arc is $d k = d s' \sin. \frac{\alpha}{2}$, therefore, $d k = \frac{12 R^3}{E l^3} H' (1 - \cos. \alpha)^2 d \alpha$. Integrating this expression, between the limits α_0 and 0, and equating the result with the value of k in Equation 5, we obtain: Mr. Shirreffs.

$$H' R = \frac{q\, l^2}{12} \frac{2 \sin. \alpha_0}{3\, \alpha_0 + \sin. \alpha_0 \cos. \alpha_0 - 4 \sin. \alpha_0} \ldots\ldots\ldots (7)$$

Now the moment, M, must cause the same angular deflection in the whole beam as the force, H'. In the general equation for angular deflection, $d\, \beta = \frac{d\, s}{x} = M \frac{d\, x}{E\, I}$, we must substitute, in one case, $H'\, R$ $(1 - \cos. \alpha)$, and in the other, M_c for M, and in both cases, $R\, d\, a$ for $d\, x$, thus getting,

$$d\, \beta = \frac{12\, H'\, R^2}{E\, l^3} (1 - \cos. \alpha)\, d\, \alpha = \frac{12\, M_c\, R}{E\, l^3}\, d\, \alpha.$$

Integrating both expressions for $d\, \beta$, between the limits α_0 and α, and equating the results, we have:

$$M_c = H'\, R \frac{\alpha_0 - \sin. \alpha_0}{\alpha_0} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

This can be readily combined with Equation 7, and the result is:

$$M_c = \frac{q\, l^2}{12} \frac{\alpha_0 - \sin. \alpha_0}{\alpha_0} \frac{2 \sin. \alpha_0}{3\, \alpha_0 + \sin. \alpha_0 \cos. \alpha_0 - 4 \sin. \alpha_0} \ldots (8a)$$

To obtain the movement of the arch under the combined action of H' and M_c, we have to substitute again in the general equation $d\, s = M\, x \frac{d\, x}{E\, I}$ (moments referred to Point X, movement of Point 2) in one case, $H'\, R\, [1 - \cos. (\alpha + \phi)]$, and in the other, M_c for M; and in both cases, $2\, R \sin. \frac{\phi}{2}$ for x, and $R\, d\, \phi$ for $d\, x$; and remembering that the movement produced by M_c is in the opposite direction to that produced by H', we have:

$$d\, s_a = \frac{24\, R^2}{E\, l^3} (H'\, R\, [1 - \cos. (\alpha + \phi)] - M_c) \sin. \frac{\phi}{2}\, d\, \phi.$$

The radial component of this movement (the arch deflection which in a curved dam will produce stresses in the vertical beams) is $d\, D'_a = d\, s'_a \cos. \frac{\phi}{2}$, therefore,

$$d\, D'_a = \frac{12\, R^2}{E\, l^3} (H'\, R\, [1 - \cos. (\alpha + \phi)] - M_c) \sin. \phi\, d\, \phi.$$

Integrating this between the limits $\phi = (\alpha_0 - \alpha) = \gamma$ and $\phi = 0$, and substituting the values of $H'\, R$ and M_c, as derived from Equations 7 and 8, we get

$$d\, D'_a = C \frac{q\, R^2}{E\, l} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which C has the general value given in Equation $9a$

$$C = \frac{2 \sin. \alpha_0}{\alpha_0} (1 - \cos. \gamma) + \frac{\sin. \alpha}{2} (2\, \gamma - \sin. 2\, \gamma)$$
$$+ \frac{\cos. \alpha}{2} (\cos. 2\, \gamma - 1) \div \frac{3\, \alpha_0}{\sin. \alpha_0} + \cos. \alpha_0 - 4 \ldots\ldots (9a)$$

Mr. Shirreffs. When $\alpha = 0$, and therefore $\gamma = \alpha_0$, the value of C is in simpler terms,

$$C_c = \frac{\frac{2 \sin. \alpha_0}{\alpha} (1 - \cos. \alpha_0) + \cos. 2 \alpha_0 - 1}{\frac{3 \alpha_0}{\sin. \alpha_0} + \cos. \alpha_0 - 4} \quad \ldots\ldots\ldots\ldots (9b)$$

and again, for the semicircle, since then $\alpha_0 = \frac{\pi}{2}$,

$$C_c = \frac{\frac{4}{\pi} - 1}{\frac{3\pi}{2} - 4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9c)$$

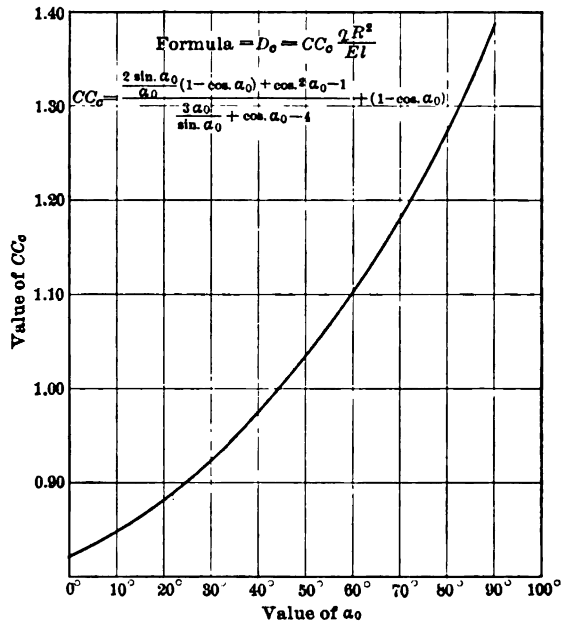


FIG. 22

The diagram, Fig. 22, prepared in accordance with the foregoing formulas, will enable the coefficient $C\ C_c$ for the total crown deflection (embracing both the effect of H' and M_c, and the axial stress), or $D_c = \frac{C\ C_c\ q\ R^2}{E\ l}$, to be obtained by inspection for an arch of any angle with reasonable accuracy. The form of Equation 9 does not permit the general value of $C\ C_a$ to be diagrammed, except for a fixed value of α_0.

It should be remarked here that, as all the deductions have been based upon the assumption that the axis of the arch is loaded with q

per unit of length, it is necessary to insert in all the equations, not the value of q as it exists at the extrados of the arch, but this value multiplied by the ratio of the radius of the extrados to the radius of the axis. It may also be noted that if it be desired to obtain the stresses at any point of the arch, it will usually be close enough to combine with the axial stress, $q\ R$, the stresses of compression or tension produced by the moment, M_a The force, H', may usually be neglected, as it has been in deriving the above formulas, as it is quite small in comparison to $q\ R$, although in very flat arches it will be necessary to take the effect of H' into account. If it be considered necessary to embrace the effect of H' upon the arch compression in the formulas, they may become considerably more complex, but will yield then smaller values of H', and therefore of D, than the foregoing. The moment at any point, α degrees from the crown, is $H'\ R\ (1 - \cos. \alpha) - M_c$, or Mr. Shirreffs.

$$M_a = \frac{q\ l^2}{6} \frac{\frac{\sin. \alpha_0}{\alpha_0} - \cos. \alpha}{\frac{3\ \alpha_0}{\sin. \alpha_0} + \cos. \alpha_0 - 4} \qquad (10)$$

M_a becomes 0, or there is a point of contrary flexure in the curved beam used as an arch under a uniform radial load, when $\cos. \alpha = \frac{\sin. \alpha_0}{\alpha_0}$, or in the semicircle when $\cos. \alpha = \frac{2}{\pi}$, or at about 50° from the crown.

Mr. Woodard's formula for obtaining the crown deflections, reduced to the notation thus far used herein, is,

$$D_c = \frac{q\ R^2}{E\ l}\ \alpha_0 \frac{\cot. \alpha_0}{2}, = C''_c \frac{q\ R^2}{E\ l},$$

and he further assumes that intermediate deflections will be proportional to the angular distance from the crown. Table No. 7 compares the deflections thus obtained in an arch having $\alpha_0 = 74°$ (Arch No. 1 in Profile No. 2 in Table No. 6) with the deflections of the same arch considered as a curved beam, the coefficients requiring to be multiplied by $\frac{q\ R^2}{E\ l}$ in both cases to obtain the actual deflections.

TABLE No. 9.

$\alpha =$	0°	14° 48'	29° 36'	44° 24'	59° 12'
CC	1.228	0.833	0.722	0.483	0.211
Ratios to CC_c	1.000	0.678	0.588	0.394	0.172
CC''	1.714	1.371	1.028	0.686	0.343
Ratios to CC''_c	1.000	0.800	0.600	0.400	0.200
Ratio $\frac{C\ C}{C\ C''}$	0.72	0.61	0.70	0.70	0.61

Mr. Shirreffs. It will be seen that, although the deflections of the curved beam are at all points much less than those resulting from the approximate method, there is no great departure from proportionality except near the crown. It will be interesting, also, to note the division of the total load between the arches and an intermediate beam. The following figures refer to the beam in Profile No. 2, which is situated 29° 36′ from the crown, the notation being that used before.

Joint.	1	2	3	4
Q	63 280	112 500	168 750	.....
P	21 420	74 900	145 210	.....
$\frac{u+v}{l}$	......	0.577	0.743	0.872

Comparing these figures with those given in the latter part of Table No. 6, it will be seen that the intermediate beams carry a greater proportion of the load than the middle beam; in other words, the deflections of the shorter beams are reduced more rapidly than the deflections of the arches. This change in the condition of the arch load from perfect uniformity, of course, will have a tendency toward a readjustment of all deflections and loads. It would have been interesting, to the writer at least, if time had permitted the working out of a profile with a practical top width and also such as to avoid tension at all points, but this has been impossible.

In all that precedes, the investigation of the problem of the curved dam has been conducted upon the supposition that the various arch slices, into which the dam is divided for calculation of movements and stresses, are perfectly free to move relatively to each other, and that only as a part of each became in turn a part of some vertical beam would there be a composite action of the masonry of the dam. But the several arch slices are not free to move relatively to each other; they react on each other in such a way as to prevent the full shortening under the axial stress due to the load which is actually carried by them. This third composite action of the masonry virtually brings into play a second set of vertical beams, resisting movement this time in the direction of the axis of the dam, instead of transversely to this axis, and resisting it more effectually, too, than the transverse beam can, because each axial beam, in the dam built in a triangular valley, is reacted upon by a beam of diminished height and therefore of greater power of resistance. But even in the case of a dam the bottom of which should be level throughout until the common vertical abutment of all the component arches was reached, a case scarcely to be met in practice, the axial beams would still exert a restraining force on the movement of the upper arch slices. The effect of this second set of vertical beams must be to transfer the effect as to average axial compression, and therefore as to deflection of each upper arch slice, of some portion of the load on this upper slice to a lower one, but at a point nearer the final point of support.

In other words, every load upon a curved dam is divided into three Mr. Shirreffs. parts, one part passing through a horizontal arch to its abutment, another part by the aid of the vertical axial beams into the abutment of some lower arch slice, and the third, and probably in an arch of moderate radius, at least as regards the loads near the top, much the smallest part, through the transverse vertical beams into their foundation.

While it may not be possible to submit this very complicated action of the several resistances of the masonry of the curved dam to a satisfactory mathematical analysis, there can be no doubt that the resistance last referred to plays a very important part in the stability of such structures. In the writer's opinion, it accounts entirely for the existence at the present time of the Zola Dam in France and the Bear Valley Dam in the United States. The former, built in 1843 and curved to a radius of 158 ft., has a height of 120 ft. and a base of 43 ft. ($b = 0.36$), while the latter, built in 1884, is curved to a radius of 335 ft., and is only 8.42 ft. thick at a point 44 ft. below the water line ($b = 0.18$). In the latter structure, especially, computation by the method herein first discussed would undoubtedly show the existence of large tensile stresses over probably the entire base. While the consideration of the axial vertical beams may not warrant the adoption of a profile which shows such stresses when thus computed, it should certainly inspire confidence in one free from tension when thus analyzed.*

As to the effect of temperature changes, unfortunately, there are few data with which to work. There is for instance an almost entire lack of information as to the penetration into the body of a massive masonry structure of the effect of the exterior seasonal changes. With the facility afforded by electrical methods for measuring such effects, it is to be desired that every engineer who builds such a structure at the present time should make the inexpensive preparation required to observe in this manner the temperature changes in the interior of the mass. The writer believes that where a radius short enough to develop the economic advantage of the curved dam can be adopted (and this seems to require a radius not much greater than 400 ft.), there need be no anticipation that temperature changes will operate to the injury of the structure.

It may be of interest to call attention to the only case, within the writer's knowledge, where the movements of a masonry dam have been actually observed. The case is that of the dam at Remscheid, Germany, built by Professor Intze, the results of such measurements being reported by him in the *Zeitschrift des Vereines deutscher In-*

* The above was written before reading the remarks of Mr. Frizell, *Proceedings*, Am. Soc. C. E., for May, 1904, p. 495, in discussing the same point. It is allowed to stand, as at least emphasizing, and perhaps making a little clearer, the action to which Mr. Frizell calls attention.

Mr. Shirreffs. *genieure* for 1895. A brief abstract of this article was given in *Engineering News* of January 30th, 1896, and the following is quoted therefrom:

"Careful measurements have been made to ascertain the radial movements * * * due to variations in pressure and temperature. The former have caused the greatest movement at the center—$1\frac{1}{16}$ ins. —while at symmetrical points between the center and the wings the observed movement was $\frac{9}{16}$ in. The greatest effect due to temperature changes was observed during a very hot and dry summer upon the curved surface, and was $\frac{7}{8}$ in. on one side of the dam against only $\frac{1}{8}$ in. on the other. This unsymmetrical action is explained by the fact that the former was entirely exposed to while the latter was largely protected against the influence of the sun. The author believes that if the dam had not been curved these movements would certainly have produced cracks."

It may be added that the dam was about 13 ft. wide at the top and 49 ft. wide at the bottom where the height was greatest (about 82 ft.), the down-stream face curving vertically between these points. The horizontal radius of the up-stream face at the top was 410 ft. It is interesting to compare the actual movement of this comparatively low dam with the deflection of only $\frac{1}{8}$ in. which Mr. Woodard computes for the Lake Cheesman Dam, assuming $E = 3\,000\,000$ lb. per sq. in.

In concluding, the writer desires to acknowledge the valuable assistance rendered him by Mr. F. F. Moore in verifying many of the deductions and computations. This assistance was the more appreciated because it was rendered through love for his profession.

Mr. Wisner. GEORGE Y. WISNER, M. AM. SOC. C. E. (by letter).—The work discussed in this paper is particularly interesting at the present time, for the reason that a number of dams are likely to be constructed in the near future, under somewhat similar natural conditions, for the storage of flood waters for irrigation purposes in the West; and especially as the structure, as completed under the present Chief Engineer, A. E. Kastl, M. Am. Soc. C. E., is one of which the engineers who designed and supervised the work, and the contractors who constructed it, may well feel proud.

Many of the large undertakings of recent years have been criticized so severely for faulty construction, both as to design and workmanship, that it is refreshing to have an actual example of completed work in every way superior to that called for in the plans and specifications under which it was built.

In regard to the rock-fill dam, originally proposed for this project, the structure would doubtless have answered the purpose for which designed if it could have been completed before being subjected to flood overflow, but, from a professional point of view, the dam, as completed, is better designed to meet the natural conditions and to give satisfactory results.

Knowing the inevitable result of permitting a heavy flood to flow over the unprotected top of a partially completed rock-fill dam, it is somewhat surprising that greater precautions were not taken. Mr. Wisner.

In the case of a rock and earth-fill dam, 50 ft. high, soon to be constructed in the Snake River, in Idaho, the plans provide for a by-pass around one end of the dam of such dimensions as will take care of the flow during construction. This by-pass, afterward, will be closed by regulating gates, for emptying the reservoir, and a thin, reinforced concrete, arched dam resting on the gate piers and abutting against the side walls of the by-pass.

It is stated in the paper that a balance valve was placed at the entrance of the lower tunnel, for the purpose of regulating the outflow, of which valve no description is given.

It is a well-known fact that many of the devices known as balance valves do not work as such in actual practice, and, as the writer is informed that the one in question has never been operated, it would be interesting to know its construction, and whether it is likely to meet the requirements.

It is inferred from the statements of the authors that the arch type of dam was adopted for the reason that such type was better adapted for the site than a straight dam, and was no more expensive, rather than from any expected additional stability to be derived from such form. However, it appears from the very interesting analysis of the stresses in the dam, given by Mr. Woodard, that at the top of the dam the arch carries nearly half the load, 6% half way down and nothing at the bottom.

When it is considered that the lower 100 ft. of this dam is practically an immense wedge of masonry held firmly in position by the solid granite of the canyon walls, preventing any tendency to slide or overturn, it is probable that this arch really takes care of a much larger percentage of the stress, which would otherwise be transmitted to the foundation, than shown by the analysis.

It is assumed in the analysis that the modulus of elasticity is the same for the vertical sections of the dam as in the horizontal rings used in computing the pressures transmitted to the side walls through the arch. In the mid-stream vertical sections of the dam, the concrete and granite boulder masonry constitutes the greater portion, while, for the upper third of the dam, where the arch is most effective, the granite masonry faces greatly increase the average modulus over that of the vertical section. The writer is strongly of the opinion that an analysis of the probable stresses in the structure, made with approximately correct moduli for the vertical and horizontal sections, would show the stability to be much greater than stated by the authors.

In localities having as wide a range of temperature as exists in most of the Western States, the stresses developed in dams from changes of temperature may, under certain conditions, exceed those

Mr. Wisner. arising from the water pressure on the face of the dam; and, where reservoirs are likely to remain only partly full for long periods, the use of steel in the upper portion of the structure is a precaution worthy of careful consideration. In the case of a straight dam, the tensile strains from change of temperature may reduce the stability to that of the theoretical section generally used in designs for gravity sections, which, with the uncertain characteristics of the materials used, cannot be considered safe. In the case of the proposed Salt River Dam, in Arizona, the upper 100 ft. of the dam will be reinforced with steel, and, as an additional precaution. the masonry of that portion of the dam will be put in only when the temperature is below the average for the year, thereby insuring either normal or compressive stresses for most of the time.

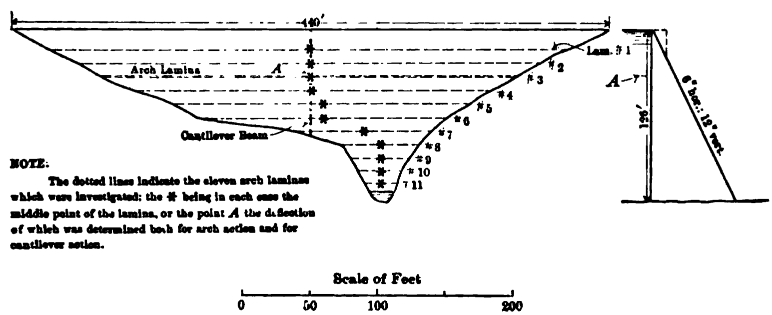
During the excessively cold weather of the winter of 1903–04, a vertical temperature crack, some 30 ft. long and ⅛ in. wide, breaking directly through the solid granite face stones, developed in the Cheesman Dam, which closed and nearly disappeared when the weather moderated. In a straight dam, with a full reservoir, such a crack would have been likely to develop leaks and, possibly, have endangered the structure, but, in the arch dam at Cheesman, it did no practical damage whatever, the crack only extending a short distance from the face.

Taking into consideration the natural conditions at the site of the Cheesman Dam and the design adopted, it is probable that a mid-stream cross-section with a base two-thirds of that used, would have been absolutely safe—in fact, the only indication of seepage that shows, with 100 ft. of water against the face of the dam, is through the narrow granite point against which the north end of the dam abuts.

In the arch type dam, such as that described, it is probably true that the rigidity of the structure will prevent it from acting as a theoretical arch, but, since the transmission of stresses through the structure, either vertically or horizontally, pre-supposes distortion, it is practically self-evident that the stresses from water pressure on the face of the dam will be distributed to the side walls by the arch, as well as to the foundation by the bending movement in the mid-stream vertical section—the amount of which will depend upon the design of the dam and the nature of the materials of which it is constructed.

The substitution of a granite masonry spillway for one in the natural rock of the granite point at the north end of the dam has strengthened the structure and improved its appearance. The completed dam is probably the best example of modern dam construction in the United States, and is well worthy of a visit by engineers interested in work of this class.

Edwin Duryea, Jr., M. Am. Soc. C. E. (by letter).—About a year ago the writer had occasion to design a masonry dam for a site especially favorable to the arch type. Economy made this type desirable, while the presence in the same State (California) of such successful examples as the Bear Valley, Sweetwater and Upper Otay Dams made it especially advisable to use the arch type if it could be shown by investigation to be theoretically justifiable. An investigation, therefore, was made with some care, in order to furnish definite grounds for the acceptance or rejection of this type. Mr. Duryea.



Figs. 23 and 24.

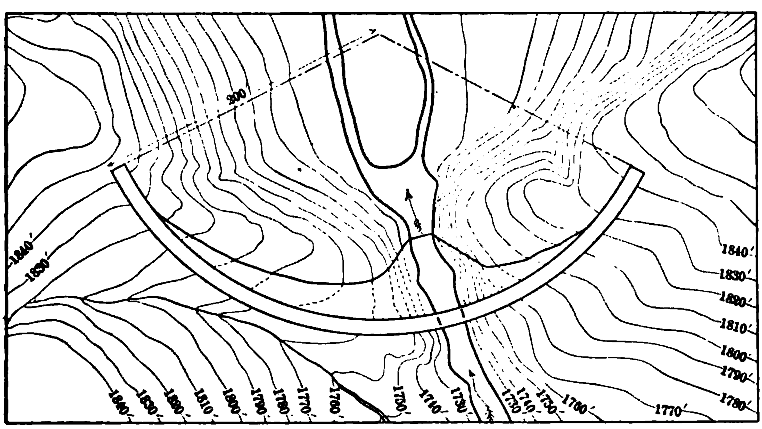


Fig. 25.

The structural plan investigated is shown by Figs. 23, 24 and 25, the principal dimensions being a length of 440 ft. on the crest, a maximum height of 126 ft., a vertical up-stream face having a radius of 200 ft., and the down-stream face battered 6 in. horizontally to 12 in. vertically. This profile, if the entire water pressure is assumed to be carried by arch action alone, gives, throughout the dam, a nearly uniform arch stress of about 12½ tons per sq. ft. of arch section.

Mr. Duryea. The investigation was made on the same general lines as given in this paper, and, while not exhaustive, showed conclusively that, for the structure and conditions assumed, only a small proportion of the total water pressure could be borne by arch action until after a sliding or giving, to develop this action, had occurred on several horizontal planes, and especially at the base of the dam.

The computed proportions of the total water pressure which could be borne by arch action before such sliding occurred vary irregularly from about 16% near the crest of the dam to zero at the base, the average at twelve points equidistant vertically being about 8 per cent. These proportions are shown in detail in Fig. 29. They are only approximations, being in each case for the middle point only of the horizontal arch lamina in question, but, at least, they show conclusively that (unless sliding has occurred) only a small amount of assistance in carrying the water pressure can be expected from the arch form. Reliance on arch action, as justifying a section thinner than a gravity section, therefore, was reluctantly abandoned.

It seems self-evident to the writer, however, that the gravity section should be built in an arch form whenever practicable, as a dam arched in plan will generally cost but little, if any, more than a straight dam of equal section, and, in case of a failure of the gravity section, the arch action will be developed and will in all probability prevent a collapse of the structure.

Rough approximate estimates of the cost of a dam at the site in question were made on five plans, as follows:

		Percentages.		
1.—Gravity type, arched in plan....	$340 000	100	152	136
2.—Sweetwater section, arch type...	230 000	68	103	92
3.—Arch type, as described..........	223 000	66	100	89
4.—Rock-fill dam	330 000	97	148	132
5.—Buttress type..................	250 000	74	112	100

The gravity type referred to is Wegmann's practical profile No. 2. The Sweetwater section is that of the Sweetwater Dam extended downward for a greater height by differences. The rock-fill dam is 10 ft. wide on the crest, with an up-stream slope of 1:1, and a down-stream slope of ¾ horizontal to 1 vertical. The buttress dam was composed of eight semi-circular arches supported by buttresses 50 ft. apart, center to center. The minimum thickness of these arches was 5 ft., increasing to 12 ft. at the lowest point of the dam by steps on the down-stream face, the up-stream face being vertical. The buttresses were 7 ft. thick at the top and increased by steps on both sides to 15 ft. thick at the base of the highest buttress.

In making these comparative estimates, the same price per cubic yard of masonry was assumed for Types Nos. 1 to 3, inclusive. The price per cubic yard assumed in estimating the rock-fill dam was one-quarter of that for Types Nos. 1 to 3; while the price assumed in connection with the buttress dam was 30% in excess of that used for Types Nos. 1 to 3. The high comparative cost of the rock-fill dam is due mainly to its bottom width being greater than the length of the narrow gorge forming the dam site, thus allowing a great spread in the base of the rock fill. The estimated cost of the buttress dam is relatively high because it was not practicable to locate it entirely on the narrow dike, as could be done with Types Nos. 1 to 3. Mr. Duryea.

In all five types considered, floods were to be taken care of by a liberal spillway, east of the dam, through the natural rock.

From the five plans compared, the buttress type was chosen because of its comparatively low cost, in conjunction with the fact that strict analysis justifies the structure as a safe one. This plan was thoroughly investigated, and full stress sheets and drawings were prepared. The maximum allowable stress in the arches from water pressure was fixed at 10 tons per sq. ft., and that in the buttresses, from a combination of water pressure and weight of masonry, at 15 tons per sq. ft. The maximum allowable shear in the buttresses was fixed at 5 tons per sq. ft. The buttresses were proportioned so as to keep the resultants of horizontal and vertical forces within the middle thirds.

While the buttress design proposed for this dam lacks conservatism, in being higher than any buttress dam yet built, no other valid objection can be made to it except the possibility (a very remote one, even in California) of its injury by earthquakes. In such a climate, no possible injury can occur from the expansion or impact of ice.

In some locations, the writer believes that the buttress type will give a dam, not only much lower in cost, but, in addition, much safer, than will the Wegmann gravity section. This section seems to be generally regarded as safe without question. It makes no provision against uplift, however, and the possibility of uplift certainly often exists. At the site described by the writer the foundation is hard, sound rock, giving an opportunity for such a good junction with the bed-rock as would leave little chance of uplift. At this site the only practical advantage of a buttress over a gravity dam, therefore, would be its saving in cost. In another dam designed by the writer, however, about 30 ft. high and having to pass floods of possibly 15 ft. in depth over the full length of its crest, the foundation was a rather soft serpentine. In this case the writer believes that a buttress dam, with its practical freedom from the possibility of uplift, would be, not only much less costly than a Wegmann section, but also much safer. In this buttress design the up-stream face was made sloping, 6.4 vertical to 10 horizontal, with the down-stream ends of the buttresses vertical. It was proposed to strengthen both the curtain and the buttresses with expanded metal and rods, though, in the computations for strength, no account was taken of this metal.

Mr. Duryea. In the following are given the assumptions and formulas by which were computed the proportions of water pressure which could be borne by the arch action of the structures shown in Figs. 23, 24 and 25. The formulas were developed and the computations made by C. B. Wing, Assoc. M. Am. Soc. C. E., who also made the computations and drawings for the two buttress designs mentioned.

In a masonry dam, any point, as A, Fig. 23, is deflected down stream as the water is allowed to rise to the top. In an arch-type dam, the displacement of the point, A, corresponds at the same time to the deflection of a cantilever beam of unit length along the dam between two transverse vertical sections, and to the deflection of an arch lamina of unit height of the dam between two horizontal planes. The deflection as a cantilever beam is caused by the bending moments induced by that portion of the water pressure carried by cantilever action, and the deflection as an arch is due to the arch contraction caused by the uniform thrust induced by that portion of the water pressure borne by arch action. Such a vertical cantilever beam and horizontal arch lamina are shown by dotted lines in Fig. 23.

The total water pressure on the dam will be shared between the two systems, arch and cantilever, directly in proportion to their respective rigidities, or inversely as their deflections. If, for any point, A, of an assumed dam section, equations be developed (1) for the deflection of the vertical cantilever beam, and (2) of the horizontal arch lamina, in each case in terms of the total unit water pressure (or weight per cubic foot of water), a relation between the proportions carried in each way will be obtained, and the amount of each proportion may be determined. The computations, at best, were very complex and tedious, and, for simplicity, the points, A, were taken at the centers only of the corresponding arch laminæ. The method used could have been applied to other points of each arch lamina, but the computations would have been still more complex, and, for the information sought, it seemed unnecessary to do so.

Assumptions.—The cantilever beams were assumed to be fixed in direction at the base, with planes before flexure still planes after flexure. The arch laminæ were assumed to be two-hinged, or changeable in direction at each bank. These two assumptions are contradictory, and both tend to reduce the apparent proportion of pressure carried by arch action. They were used for simplicity, however, and as being sufficiently exact for the end sought.

Nomenclature.—(See Figs. 26 and 27).

1.—Vertical section through the point, A:

Let h = height of dam from base (below A) to crest;

b = thickness of dam at base (below A),

x = height from base of dam to any horizontal section;

f = thickness of dam at height, $x = \frac{1}{2}(h - x)$.

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2.—Horizontal section through the point, A :

Let s = span of horizontal arch lamina before being shortened by pressure;

r = mean radius of horizontal arch lamina before being shortened by pressure;

l_1 = chord of half of horizontal arch lamina before pressure is applied;

l_2 = chord of half of horizontal arch lamina after pressure is applied;

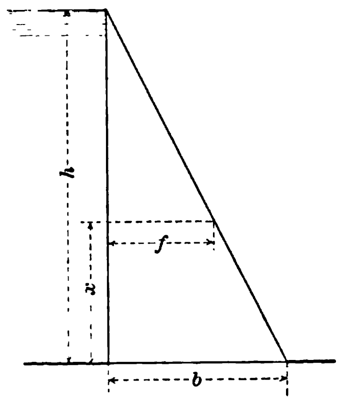

FIG. 26.

T = thrust on horizontal arch lamina due to proportion of water pressure borne by it;

e = total shortening of chord, l_1, due to the thrust, T;

y_1 and y_2 = ordinates corresponding to l_1 and l_2;

$d = y_1 - y_2$ = deflection of the point, A.

3.—Water pressures:

Let k = the weight of a cubic foot of water = 62.5 lb.;

k_g = proportion of the unit weight of water (number of 62.5ths) which will be borne by gravity action;

k_a = proportion of the unit weight of water (number of 62.5ths) which will be borne by arch action.

4.—Compression of masonry:

Let E = modulus of elasticity of masonry = 1 500 000 lb. per sq. in.

The value of the modulus of elasticity of masonry is very variable and uncertain, and, therefore, it would be much better, theoretically, to eliminate this quantity from the final equation. This was not done, however, but it appears in both terms of the equation, and its effect, as will be seen later, is thus practically eliminated.

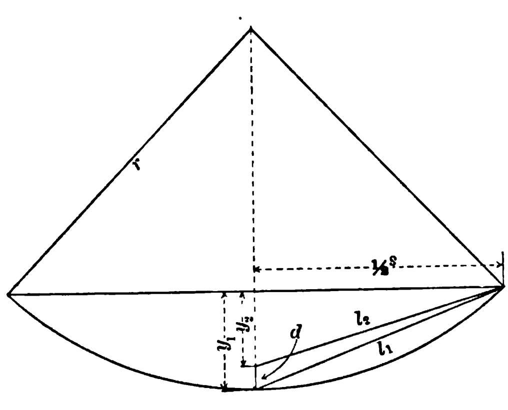

FIG. 27.

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Development of Equation.—By the ordinary theory of flexure, the deflection, d, of any point, A, in a vertical section of the dam of unit length (along the crest), acting as a cantilever beam and subjected to that proportion of the total water pressure which can be borne by cantilever action, will be given by the formula,

$$d = \frac{h^3\ k_g}{b^3\ E} x^2,$$

or

$$\frac{8\ k_g}{E} x^2$$

for the section under consideration.

This formula is derived as follows: While, theoretically, the formula may not be exactly correct, the results obtained from it must be a very close approximation.

The differential equation of the elastic curve is

$$\frac{d^2\ y}{d\ x^2} = \frac{M}{E\ I},$$

where M = moment of the external forces;

and I = moment of inertia of the section of beam.

The cantilever beam is formed of a transverse section of the dam, of unit length (along the crest), and is shown, with the water pressure acting upon it, in Fig. 28.

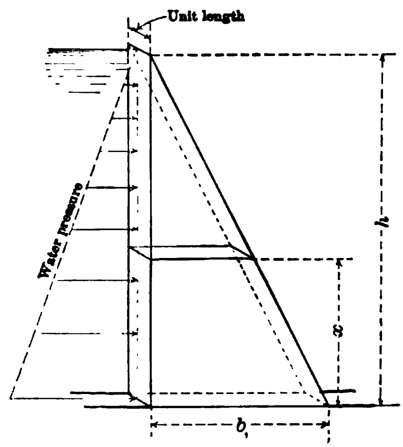

FIG. 28.

The moment of the forces acting above any section, x linear units from the bottom, is

$$M = \frac{(h - x)\ k}{2}\ (h - x)\ \frac{h - x}{3} = \frac{(h - x)^3}{6} - k,$$

where k = the weight of a cubic unit of water.

The moment of inertia of the rectangular horizontal section at is

$$I = \frac{\text{breadth} \times \text{height}^3}{12}, \text{ or, as the breadth is unity,}$$

$$I = \frac{\text{height}^3}{12}$$

$$= \frac{\left(\frac{h - x}{h}\ b_1\right)^3}{12} = \frac{b_1^3}{12\ h^3}\ (h - x)^3,$$

and

$$\frac{d^2\ y}{d\ x^2} = \frac{M}{E\ I} = \frac{12\ (h - x)^3\ k\ h^3}{6\ (h - x)^3\ b_1^3\ E} = \frac{2\ h^3\ k}{b_1^3\ E}.$$

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Integrating,

$$\frac{d\,y}{d\,x} = \frac{2\,h^3\,k}{b_1{}^3\,E}\,x + (C_1 = 0),$$

and again,
$$y = \frac{h^3\,k}{b_1{}^3\,E}\,x^2 + (C_2 = 0),$$

and finally, as $h = 2\,b_1$,

$$y = \frac{8\,k}{E}\,x^2.$$

Replacing y by the symbol for deflection, d, and k by k_g, the proportion of the water pressure borne by cantilever action, the equation becomes,

$$d = \frac{8\,k_g}{E}\,x^2, \text{ as given.}$$

The deflection of the horizontal arch lamina at the point, A, is more difficult to obtain. An approximate determination of the deflection at the center of a flat arch may be made as follows:

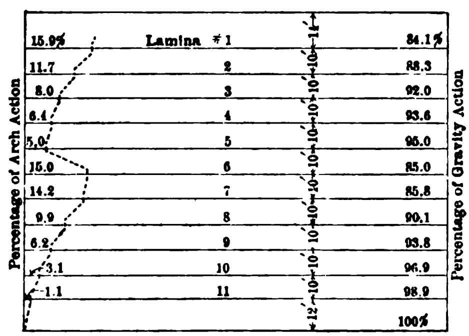

FIG. 29.

Assume the deflection, d, to be due to a decrease in the length of the chord, l_1 (Fig. 27), the length becoming l_2 after the load is applied;

$$d = y_1 - y_2,$$

$$y_1 = r - \sqrt{r^2 - \frac{s^2}{4}};$$

let
$$\sqrt{r^2 - \frac{s^2}{4}} = a,$$

then
$$y_1 = r - a$$

$$l_1 = \sqrt{y_1{}^2 + \frac{s^2}{4}} = \sqrt{r^2 - 2\,r\,a + a^2 + \frac{s^2}{4}}$$

$$l_2 = l_1 - e,$$

and, since the arch lamina is of unit thickness vertically,

$$e = \frac{T}{f\,E}\,l.$$

The thrust or compressive force in any arch lamina, x, will be caused by the water pressure, $k_a\,(h - x)$, at that lamina, and will be

$$T = k_a\,(h - x)\,r.$$

Therefore,
$$e = \frac{k_a\,(h - x)\,r}{f\,E}\,l^1;$$

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and
$$l_2 = l_1 \left(1 - \frac{k_a\,(h - x)\,r}{f\,E}\right);$$
let
$$\left(1 - \frac{k_a\,(h - x)\,r}{f\,E}\right) = c,$$
then
$$l_2 = l_1\,c.$$
$$y_2 = \sqrt{l_2^2 - \frac{s^2}{4}} = \sqrt{l_1^2\,c^2 - \frac{s^2}{4}}$$
$$= \sqrt{\left(r^2 - 2\,r\,a + a^2 + \frac{s^2}{4}\right) c^2 - \frac{s^2}{4}}$$

The point, A, is a material point and can have but a single deflection at any one time. The deflection in the arch lamina at this point, therefore, must necessarily be the same as that in the cantilever beam. Then,
$$d = y_1 - y_2 = \frac{8\,k_g}{E}\,x^2$$
and substituting values of y_1 and y_2,
$$(r - a) - \sqrt{\left(r^2 - 2\,r\,a + a^2 + \frac{s^2}{4}\right) c^2 - \frac{s^2}{4}} = \frac{8\,k_g}{E}\,x^2.$$

Substituting in this equation the values of a and c, and replacing k_g by its equivalent, $(k - k_a)$, or $(62.5 - k_a)$, the equation is as follows:
$$r - \sqrt{r^2 - \frac{s^2}{4}} - \sqrt{\left[r^2 - 2\,r\sqrt{r^2 - \frac{s^2}{4}} + \left(r^2 - \frac{s^2}{4}\right) + \frac{s^2}{4}\right] \times \left[1 - \frac{k_a\,(h - x)}{f\,E}\,r\right]^2 - \frac{s^2}{4}}$$
$$= \frac{8\,(62.5 - k_a)}{E}\,x^2,$$
which gives, after simplification, as the final equation,
$$r - \sqrt{r^2 - \frac{s^2}{4}} - \sqrt{\left[\frac{s^2}{4} + \left(r - \sqrt{r^2 - \frac{s^2}{4}}\right)^2\right] \times \left[1 - \frac{k_a\,(h - x)}{f\,E}\,r\right]^2 - \frac{s^2}{4}}$$
$$= \frac{500 - 8\,k_a}{E}\,x^2.$$

After the selection of any particular arch lamina, the only unknown quantity in this equation is k_a, or the number of pounds per cubic foot of water which can be borne by arch action.

The solution of this equation involved a great amount of labor, as seven-place logarithmic tables were not sufficiently accurate to give correct results, and it was found necessary to make the computations by ordinary multiplication. The proportions borne by arch action were computed laboriously by this method for three arch laminæ, the data and results being given in Table 8. An extended investigation* of arch dams then came to the writer's attention, and, by a formula for arch deflection found therein, the proportions borne by arch action, in the cases of the three laminæ already investigated and in eight others, were computed in a few hours. Mr. Duryea.

TABLE No. 8.—Data and Results.

Numbers of arch laminæ.	Data, in Feet.						Results, in 62.5ths, and in Percentages.		
	r	s	h	x	b	f	k_a	k	kg
1	195	309	74	60	37	10	9.96 = 15.9%	62.5 = 100%	84.1%
2	194	275	74	50	37	12	*7.34 = 11.7	" "	88.3
3	191.5	241	74	40	37	17	5.00 = 8.0	" "	92.0
4	189	192	79	35	39.5	22	4.00 = 6.4	" "	93.6
5	186.5	146	84	30	42	27	3.1 = 5.0	" "	95.0
6	184	98	119	55	59.5	32	9.4 = 15.0	" "	85.0
7	181.5	67	126	52	63	37	*8.9 = 14.2	" "	85.8
8	179	53	126	42	63	42	6.2 = 9.9	" "	90.1
9	176.5	44	126	32	63	47	3.9 = 6.2	" "	93.8
10	174	34	126	22	63	52	1.95 = 3.1	" "	96.9
11	171.5	25	126	12	63	57	*0.67 = 1.1	" "	98.9

* By the longer method, k_a = 7.4, 8.8 and 0.06 for Laminæ Nos. 2, 7 and 11, respectively.

The formula in question may be found on page 82 of Vischer and Wagoner's paper, and is as follows:

$$e = d\,V,$$

in which

e = the total shortening under the arch pressure of half the arch lamina;

d = the deflection of the center point of the arch lamina;

V = one-quarter of the total angle (in terms of the radius) subtended by the arch lamina.

From the theory of compression of elastic bodies,

$$e = \frac{T}{f\,E}\,l = \frac{k_a\,(h - x)\,r}{f\,E}\,l,$$

where the vertical thickness of the arch lamina is unity and the nomenclature is as before, except that l now represents the half length of the arch lamina instead of the length of the half chord.

For the profile being investigated,

$$f = \tfrac{1}{2}\,(h - x)$$

and

$$e = \frac{2\,k_a\,r}{E}\,l.$$

* Vischer and Wagoner, "On the Strains in Curved Masonry Dams," *Transactions*, Tech. Soc. Pacific Coast, Vol. VI, Dec., 1889, pp. 75-151.

Mr. Duryea.

Therefore, $$d = \frac{e}{V} = \frac{2\,k_a\,r}{E\,V}\,l;$$

but $$2\,V\,r = l, \text{ or } V = \frac{l}{2\,r}.$$

Therefore, $$d = \frac{2\,k_a\,r}{E\,V}\,l = \frac{2\,k_a\,r}{E\,\frac{l}{2\,r}}\,l = \frac{4\,k_a\,r^2}{E},$$

as an expression for the deflection of the center point of an arch lamina.

As before, the deflection of the same point, in terms of the deflection of a vertical cantilever beam, is

$$d = \frac{8\,k_g}{E}\,x^2 = \frac{500 - 8\,k_a}{E}\,x^2.$$

The deflection of the arch lamina and that of the cantilever beam must be the same.

Therefore, $$\frac{4\,k\,r^2}{E} = \frac{500 - 8\,k_a}{E}\,x^2,$$

and $$k_a\,r^2 = (125 - 2\,k_a)\,x^2 = 125\,x^2 - 2\,x^2\,k_a,$$
$$k_a\,(r^2 + 2\,x^2) = 125\,x^2,$$

and, finally, $$k_a = \frac{125\,x^2}{r^2 + 2\,x^2}.$$

By this formula the proportion of the total water pressure carried by arch action was computed for eleven arch laminæ, and the results obtained are given in Table No. 8. The close correspondence between the values as computed by the two methods is notable, and seems to be a check, not only on the correctness of the methods and computations, but also to show that the choice (in connection with the longer method) of a value for the modulus of elasticity of masonry has but a small effect on the resulting computed proportion of arch action.

The method of Messrs. Vischer and Wagoner, of course, is much to be preferred, both theoretically and practically. Their valuable study of arch dams seems to be too little known to the profession.

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXX. No. 7.

SEPTEMBER, 1904.

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1904.

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Vol. XXX. SEPTEMBER, 1904. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

September 7th, 1904.—The meeting was called to order at 8.45 P. M.; C. C. Schneider, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 96 members and 21 guests.

The minutes of the meeting of June 1st, 1904, were approved as printed in the *Proceedings* for August, 1904.

A paper, entitled "The Installation of a Pneumatic Pumping Plant," by Arthur H. Diamant, Jun. Am. Soc. C. E., was presented by the author. A written communication from Elmo G. Harris, M. Am. Soc. C. E., was presented by the Secretary, and the subject was discussed by Edward Wegmann, M. Am. Soc. C. E., who illustrated his remarks with lantern slides.

A second paper, entitled "Some Notes on the Creeping of Rails," by Samuel Tobias Wagner, M. Am. Soc. C. E., was presented by the author. Written discussions on this paper, by Messrs. Gustav Lindenthal, W. M. Camp, F. S. Stevens, Hunter McDonald and P. H. Dudley, were presented by the Secretary.

Ballots for membership were canvassed, and the following candidates elected:

As Members.

Fred Walter Abbott, Philadelphia, Pa.
Albert Morrill Blodgett, Kansas City, Mo.
Samuel Dunlap Brady, Parkersburg, W. Va.
David Coe, Rincon Antonio, Oaxaca, Mexico.
Frederick Reginald French, Santa Barbara, Cal.
Harry Fuller, Cleveland, Ohio.
Clarence Thomas Johnston, Cheyenne, Wyo.
Ernest William Moir, London, England.
Paul N Nunn, Niagara Falls, N. Y.
Carl Redlich, Vienna, Austria.
Wallace Berkley Riegner, Philadelphia, Pa.
Glenn Mason Scofield, Philadelphia, Pa.

As Associate Members.

George Henry Bliss, North Yakima, Wash.
George William Booth, Weston, Mass.
Sherman Worcester Bowen, St. Louis, Mo.
Benjamin Thomas Buffinton, Fall River, Mass.
Edward Smith Cole, Upper Montclair, N. J.
Guy Whitmore Culgin, New York City.
Robert Hawxhurst, Jr., Hilo, Hawaii.
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Ernest Avery Lamb, Albany, N. Y.
Thomas Monahan Lavelle, Ambridge, Pa.
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Robert Austen McCulloch, New York City.
Harry Sherwood Royden McCurdy, Boston, Mass.
Henry Gorton Opdycke, New York City.
Walter Woodbury Patch, South Framingham, Mass.
Clarence Hard Thompson, Syracuse, N. Y.
Constantine Borisson Voynow, Philadelphia, Pa.
Parley Lycurgus Williams, Jr., Bingham, Utah.

As Associate.

John Alexander Dailey, Topeka, Kans.

The Secretary made the following announcements:

The transfer of the following candidates, by the Board of Direction, on September 6th, 1904:

FROM ASSOCIATE MEMBER TO MEMBER.

CHARLES AMES ALDEN, Steelton, Pa.
WILLIAM ANDREW ALLEN, New York City.
JAMES HENRY BRACE, New York City.
JOHN THOMPSON EASTWOOD, New Orleans, La.
RUTGER BLEECKER GREEN, Detroit, Mich.
VAN ALEN HARRIS, San Juan, Porto Rico.
ROBERT HOFFMANN, Cleveland, Ohio.
ALLAN APPLETON ROBBINS, New York City.
OSCAR EMIL STREHLOW, South Bend, Ind.

The election of the following candidates, by the Board of Direction, on September 6th, 1904:

AS JUNIORS.

NATHANIEL TOWNSEND BLACKBURN, Galveston, Tex.
CHARLES TARBELL DUDLEY, San Francisco, Cal.
ARTHUR ROBERT EITZEN, Columbia, Mo.
HENRY HEYWOOD FOX, Cambridge, Mass.
ROBERT ELLIOT HALL, Auburn, N. Y.
JOHN HAWKESWORTH, New York City.
FRAZER CROSWELL HILDER, Washington, D. C.
CLEMENT JOHN HOWARD, Galveston, Tex.
JACOB BACON HUTCHINGS, Jr., Louisville, Ky.
LUTHER ELMAN JOHNSON, Lawton, Okla.
GUSTAVE EDMUND KAHN, Milwaukee, Wis.
FRANK CECIL MAGRUDER, Belle Fourche, S. Dak.
FRANK LESLIE WILCOX, St. Louis, Mo.

The Secretary announced the following deaths:

GEORGE CURTIS TINGLEY, elected Member, September 6th, 1871; died April 30th, 1904.

THOMAS MCKEOWN, elected Member, December 3d, 1879; died June 7th, 1904.

RICHARD CALVIN MCCALLA, elected Junior, July 2d, 1890; Associate Member, September 2d, 1891; Member, December 5th, 1894; died June 13th, 1904.

ALONZO J. TULLOCK, elected Member, June 6th, 1883; died July 21st, 1904.

JOSEPH NORTON GREENE, elected Member, October 5th, 1887; died August 10th, 1904.

George Clinton Gardner, elected Member, November 3d, 1875; died August 12th, 1904.

Fritz Carl Anders Georg Berggengren, elected Associate Member, December 2d, 1896; died August 8th, 1904.

The Secretary announced the details of the entertainment of the visiting Members of the Institution of Civil Engineers to be held from September 13th to 16th, 1904.

Adjourned.

September 21st, 1904.—The meeting was called to order at 8.40 P. M., George H. Pegram, Director, in the chair; Chas. Warren Hunt, Secretary, and present, also, 135 members and 46 guests.

Three papers were presented for discussion: The first, entitled "General Methods for the Calculation of Statically Indeterminate Bridges, as used in the Check Calculations of Designs for the Manhattan Bridge and the Blackwell's Island Bridge, New York," by Frank H. Cilley, S. B., was presented by the author. The second paper, entitled "Theory and Formulas for the Analytical Computation of a Three-Span Suspension Bridge with Braced Cable," by Leon S. Moisseiff, Assoc. M. Am. Soc. C. E., was presented by the author, and a written discussion by I. P. Church, Assoc. Am. Soc. C. E., was read by title by the Secretary. The third paper, entitled "A Rational Form of Stiffened Suspension Bridge," by Gustav Lindenthal, M. Am. Soc. C. E., was presented by the author, and was discussed by Messrs. W. Hildenbrand, Joseph Mayer, R. S. Buck, W. W. Crehore and Gustav Lindenthal.

The Secretary announced the following deaths:

Standish Barry Burton, elected Member, June 1st, 1898; died August 13th, 1904.

Reuben Shirreffs, elected Member, June 4th, 1890; died August 31st, 1904.

William Arthur Pratt, elected Member, December 4th, 1895; died September 19th, 1904.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

September 6th, 1904.—8.40 P. M.—Vice-President Deyo in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Buck, Craven, Croes, Ellis, Gowen, N. P. Lewis, Noble, Osgood and Pegram.

Routine business only was considered.

Seventy-six applications for admission were considered and acted upon.

Nine Associate Members were transferred to the grade of Member.

Six applications for transfer to a higher grade were declined.

Thirteen candidates for Junior were elected.

Adjourned.

ANNOUNCEMENTS.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

MEETINGS.

Wednesday, October 19th, 1904.—At this meeting a paper by C. C. Schneider, M. Am. Soc. C. E., entitled "The Structural Design of Buildings" will be presented for discussion.

This paper is printed in this number of *Proceedings.*

INTERNATIONAL ENGINEERING CONGRESS AND THIRTY-SIXTH ANNUAL CONVENTION OF THE SOCIETY.

As previously announced, in the Programme of the **International Engineering Congress,** which has been issued to all members of the Society, there will be meetings of the **Congress** at 10 A. M. each day, beginning Monday, October 3d, and continuing throughout the week, the last meeting of the Congress being held on Saturday morning, October 8th. **Members of the Society in all grades are Members of the Congress.**

The **Thirty-Sixth Annual Convention** of the **Society** will be held on the afternoon of Monday, October 3d, in the Hall of Congresses, Louisiana Purchase Exposition, at 2.30 P. M. At this meeting the President will deliver the Annual Address. At the close of the address of the President, the business meeting required by the Constitution will be held. A meeting of the Board of Direction, as required by the Constitution, will be held at a time to be subsequently determined.

A circular, relating to hotel accommodations, which was prepared by the Local Committee of the Society in St. Louis, has been issued to all members, and in this the "Inside Inn" was recommended as, all things considered, the most available for members and their families.

In this circular it is explained that it is impossible to secure a lower rate for transportation than the Exposition excursion rate established by all railroads, this rate being less than the usual, one fare and one-third, round trip, convention rate.

To all members of the Society who have, in response to the circular already issued, specified the subjects it is their purpose to discuss, copies of such of the papers as have been printed in advance form have already been forwarded, and, upon request, the Secretary will be glad to forward such copies to members who have not already asked for them.

Owing to the large number of papers, which cover many subjects, it will be impossible to forward Advance Copies of all papers to all members of the Congress, and it has been necessary to restrict the issue of Advance Copies to those who expect to present discussions either in person at the Congress or by letter.

INTERNATIONAL ENGINEERING CONGRESS.

CHAIRMAN:

CHARLES HERMANY, President, Am. Soc. C. E.

CHAIRMEN OF SECTIONS:

Section A.—Waterways.

ALFRED NOBLE, Past-President, Am. Soc. C. E.

Section B.—Municipal.

J. JAMES R. CROES, Past-President, Am. Soc. C. E.

Section C.—Railroads.

ROBERT MOORE, Past-President, Am. Soc. C. E.

Section D.—Materials of Construction.

FREDERIC P. STEARNS, M. Am. Soc. C. E.

Section E.—Mechanical.

WILLIAM METCALF, Past-President, Am. Soc. C. E.

Section F.—Electrical.

FRANK J. SPRAGUE, M. Am. Soc. C. E.

Section G.—Military and Naval.

WILLIAM P. CRAIGHILL, Brig.-Gen. U. S. A. (retired);
Past-President, Am. Soc. C. E.

Section H.—Miscellaneous.

OCTAVE CHANUTE, Past-President, Am. Soc. C. E.

SECRETARY:

CHARLES WARREN HUNT, *Secretary*, Am. Soc. C. E.

COMMITTEE IN CHARGE OF ENGINEERING CONGRESS.

Henry S. Haines, *Chairman*, Detroit, Mich.
L. F. G. Bouscaren, *Vice-Chairman*, Cincinnati, Ohio.
Charles Warren Hunt, *Secretary*, New York City.
Joseph M. Knap, *Treasurer*, New York City.

William Metcalf, Pittsburg, Pa.
William P Craighill, Charleston, W. Va.
M. T. Endicott, Washington, D. C.
Joseph Ramsey, Jr., St. Louis, Mo.
Robert W. Hunt, Chicago, Ill.
L. B. Stillwell, New York.
George B. Post, New York.
Joseph O. Osgood, Jersey City, N. J.
Benjamin M. Harrod, New Orleans, La.
George W. Melville, Philadelphia, Pa.
Elmer L. Corthell, New York.
Hunter McDonald, Nashville, Tenn.
Palmer C. Ricketts, Troy, N. Y.
M. L. Holman, St. Louis, Mo.
Alfred Craven, New York.
George H. Pegram, New York.
George S. Davison, Pittsburg, Pa.

COMMITTEE OF ARRANGEMENTS FOR THE ANNUAL CONVENTION.

George H. Pegram, *Chairman;*
Alfred Craven, George S. Davison,
Joseph O. Osgood, Hunter McDonald,
Chas. Warren Hunt, *Secretary.*

UNIVERSAL EXPOSITION, ST. LOUIS, 1904.

The Society has undertaken to provide for an engineering exhibit, and the establishment of Headquarters for visiting engineers **in the center of the Liberal Arts Building,** and the Board of Direction has appropriated sufficient funds to defray the necessary expense.

This matter is in the hands of the following Committee:

Robert Moore, M. Am. Soc. C. E., St. Louis, Mo., *Chairman.*
Edward C. Carter, M. Am. Soc. C. E., Chicago, Ill.
Mordecai T. Endicott, M. Am. Soc. C. E., Washington, D. C.
James L. Frazier, " " Frankfort, Ind.
William Jackson, " " Boston, Mass.
Emil Kuichling, " " New York, N. Y.
J. L. Van Ornum, " " St. Louis, Mo.
John F. Wallace, " " Chicago, Ill.
O. E. Mogensen, *Sect'y,* " " St. Louis, Mo.

PRIVILEGES OF LOCAL SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

The **Boston Society of Civil Engineers** will welcome any member of the American Society of Civil Engineers at its library and reading room, 715 Tremont Temple, Boston, which is open on week days from 9 A. M. to 5 P. M. Members will also be welcome at the meetings which are held in the same building on the evenings of the fourth Wednesday in January, and the third Wednesdays of other months, except July and August.

The rooms of the **St. Louis Engineers' Club**, in the business center of St. Louis, will be kept open during the World's Fair season, May 1st to December 1st, 1904, and visiting engineers are cordially invited to use them for mail, telephone service, information, etc.

The courtesies of the **Engineers' Society of Western Pennsylvania** have been extended to members of the American Society of Civil Engineers. The rooms of the Society, 410 Penn Ave., Pittsburg, Pa., are open at all times, and meetings are held as follows, except during July and August. REGULAR SECTION, Third Tuesdays; CHEMICAL SECTION, Thursdays following third Tuesdays; MECHANICAL SECTION, first Tuesdays; STRUCTURAL SECTION, Fourth Tuesdays.

The Western Society of Engineers, Monadnock Block, Chicago, Ill., tenders to members of this Society the use of its rooms and facilities, together with the good offices of its Secretary and of a special committee appointed for that purpose.

The Civil Engineers' Club of Cleveland, Ohio, invites members of this Society to make use of the Club rooms, at any time when in Cleveland. Cards will be furnished on application to the Secretary, Mr. J. C. Beardsley.

SEARCHES IN THE LIBRARY.

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many such searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling, compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

Copies of all lists of references are filed, so that in many cases it is only necessary to make a typewritten copy, which reduces the cost of searches to a minimum.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

ACCESSIONS TO THE LIBRARY.

From August 7th to September 13th, 1904.

DONATIONS.*

TYPES AND DETAILS OF BRIDGE CONSTRUCTION.

By Frank W. Skinner, M. Am. Soc. C. E. Cloth, 9 x 7 in., 6 + 294 pp. New York, McGraw Publishing Company, 1904.

The author states that it is his purpose to present the development of advanced practice, in bridge construction, and its standard details, to illustrate the classes of structures adapted to different conditions, show some of the characteristic differences between American and foreign design and illustrate some primitive or obsolete constructions, recording important and well-known examples. The bridges have been arranged in order and grouped in classes, and the descriptions are in most cases supplemented by specific references to any more extended articles which have been published about them in technical journals or professional papers, and may be consulted in libraries for additional data in special cases. The book is divided into four parts: Part I, Wood and Iron Arch Spans; Part II, Spandrel Braced Arches; Part III, Arch Trusses; and Part IV, Plate Girder Arches. It has an alphabetical index of bridges, and a classified index of bridge details; both indexes are included in four pages.

BROWN'S DIRECTORY OF AMERICAN GAS COMPANIES.

Gas Statistics, 1904. Compiled and corrected by E. C. Brown. Cloth, 10 x 7 in., 272 pp. New York, Press of the Progressive Age, 1904. $5.00.

This book is published annually, and contains statistics of gas companies in the United States, Canada and South America, and of electrical companies where they are operated under the same name as the gas companies. Acetylene and gasolene gas plants are included, and also a list of parent or operating companies. There is a list of officers and members of the gas associations in the United States, an analysis of British gas accounts for 1902–03, and a bibliography for gas men, divided by subject. The arrangement of the whole book is alphabetical, but it also has a one-page index of contents.

TRANSPORTATION BY RAIL.

An Analysis of the Maintenance and Operation of Railroads, Showing the Character and Cost of the Service Performed by Railway Companies in the Maintenance of Highways for Commerce, and as Common Carriers of Passengers, Freight and the United States Mails over such Highways. By T. M. R. Talcott. Cloth, 8 x 7 in., 84 pp. Richmond, Va., Whittet and Shepperson, 1904. (Donated by the author.)

The preface states that the statistics given in this book are based on the many years' experience of the author in the construction, maintenance and operation of railways, during which time he had the accounts kept to suit his special method of analyzing the cost of maintenance and operation. This method of analysis was applied to a number of Southern roads for periods of from three to ten years, and the units of cost ascertained differed somewhat for the same service, according to conditions on different roads, such as standards of construction and equipment, varying rates of pay for employees and diversity in the cost of materials and supplies. The road selected as an illustration seemed to represent the average of Southern roads of the same general standard. The statement is made that the present method of accounting, as established by the Interstate Commerce Commission, does not admit of the application of this method of analyzing operating expenses, and therefore the present cost of service cannot be accurately given; but the general managers who wish to know the exact cost of each unit of service on their roads may have their accounts kept so as to admit of this method of analysis.

CHART METHOD OF REDUCING POLARIS OBSERVATIONS.

By Clark Brown. Cloth, 14 x 14 in., 5 pp. Albany, Published by the Author, 1904. Paper, $1.00; cloth, $1.50. (Donated by the author.)

The author states that, having found computations by the usual methods tedious when the observations were made at convenient times in the early evening, he has been led to devise a Chart Method for making the reductions. In preparing this method for publication it was thought best to accompany it with a table of the time of the culmination of Polaris, on a new plan, and also with a set of simple directions for making the observations. The table and chart are prepared from data found in the United States Government publications, and, while not of sufficient refinement for use in extensive geodetic surveys upon which the larger theodolites are used, the author believes them to be as accurate as will be required for use with the engineer's transit, the probable error of the azimuth line being less than the probable error in the measurement of an angle.

THE INDUSTRIAL AND ARTISTIC TECHNOLOGY OF PAINT AND VARNISH.

By Alvah Horton Sabin. Cloth, 9 x 6 in., 6 + 372 pp., illus. New York, John Wiley and Sons; London, Chapman and Hall, Limited, 1904. $3.00.

The writer says that it is his aim to give a correct general outline of the subject of Paints and Varnishes, with a brief account of their modern use and of the principles which are involved in their fabrication and application. It is also stated that many of the facts herein noted, though old, are practically unknown, and some of them exactly anticipate recently patented processes; their value to the public in that way is sufficient excuse for their republication. Scarcely any patents in this line are of any value or validity; and the "secret processes" which are continually vended are for the most part neither secret nor new. The only trade secrets lie in the incommunicable intimate knowledge of the expert, and are made valuable only by his unceasing care, vigilance and conscientiousness. Theories, however, may be made known, and the attention of the student may be intelligently directed to their application. The book contains an alphabetical index of eight pages.

Gifts have also been received from the following:

Aachen Tech. Hochschule. 8 pam.
Abner Doble Co. 1 pam.
Am. Soc. of Mech. Engrs. 1 pam.
Bell, George Joseph. 1 bound vol.
Borxovsxi, J. Th. 1 vol., 1 map, 1 bound vol.
Bradley, C. W. 1 vol
Brit. Fire Prevention Committee. 2 pam.
Buffalo, Rochester & Pittsburgh Ry. Co. 1 pam.
Cal. Acad. of Sci. 1 vol.
Canadian Inst 1 vol.
Castanheira das Neves, J. da P. 1 pam.
Central of Georgia Ry. Co. 1 pam.
Chicago Terminal Transfer R. R. Co. 1 pam.
Comitato "Pro Roma Maritima." 2 pam.
Comstock, C. W. 1 pam.
Cuba—Secretario de Obras Publicas. 4 vol.
Denver & Rio Grande R. R. Co. 1 pam.
Fisk, W. L. 1 pam., 2 maps.
Floesch, J. M. 1 pam.
Gila Valley Globe and Northern Ry. Co. 1 pam.
Glasgow & South-Western Ry. Co. 1 pam.
Gould, E. S. 1 vol.
Great Britain—Patent Office. 7 vol., 11 pam.
Hannover Königliche Tech. Hochschule. 1 vol.
Haverhill, Mass.—City Engr. 1 pam.
India—Pub. Works Dept. Rys. 1 vol.
Inst. of Civ. Engrs. 2 bound vol.
Inst. of Civ. Engrs. of Ireland. 1 vol.
Inst. of Min. and Metallurgy. 2 bound vol.
Iowa—R. R. Commrs. 1 bd. vol.
Iron and Steel Inst. 3 bound vol., 1 pam.
Jackson, Granberry. 1 pam.
Jour. of the Royal United Service Inst. 4 nos.
Königliche Tech. Hochschule zu Berlin. 1 pam.
Limbeck, Zdenko Ritter von. 1 pam.
Liverpool Eng. Soc. 1 vol.
McGill Univ. 1 bound vol.
McGraw Pub. Co. 1 vol.
Manchester Assoc. of Engrs. 1 bound vol.
Met. Board of Water Supply. 1 pam.
Mich. Coll. of Mines. 4 pam.
Midland Ry. Co. 1 pam.
New England Cotton Mfgrs. Assoc. 1 bound vol.
New England Water Works Assoc. 1 vol.
New South Wales—Govt. Statistician. 1 bound vol.
New York, Ontario & Western Ry. Co. 1 pam.
North. E. P. 1 pam.
Northampton, Mass.—City Clerk. 1 bound vol.
Patschke, A. 1 pam.
Peabody Coal Co. 1 pam.
Pharr, H. N. 2 pam.
Royal Soc. of Canada. 1 bound vol.
Smith, J. A. 1 pam.
Soc. Belge des Ingenieurs et des Industriels. 8 pam.
Toronto, Ont.—City Engr. 1 bound vol.
Tyrrell, H. G. 1 pam.
U. S. Bureau of Commerce & Labor. 1 vol.
U. S. Bureau of Forestry. 7 pams.
U. S. Bureau of Statistics. 1 vol., 1 pam.
U. S.—Director of the Mint. 14 bound vol., 5 vol., 2 pam.
U. S. Div. of Foreign Markets. 1 vol., 2 pam.
U. S. Office of Exper. Station. 3 pam.
U. S. War Dept. 14 specif.
United Water Impvt. Co. 1 pam.
Univ. of Ill. 1 vol.
Univ. of Wis. 1 pam.
West Va. Geol. Surv. 1 bound vol.
Western N. Y. Car Service Assoc. 1 pam.
Western Soc. of Engrs. 1 pam.
Woburn, Mass.—City Clerk. 1 bound vol.

BY PURCHASE.

A Treatise on Surveying. Comprising the Theory and the Practice. Part II Higher Surveying. By William M. Gillespie. Revised and enlarged by Cady Staley. New York, D. Appleton & Company, 1902.

Die Vagabundierenden Ströme Elektrischer Bahnen. By Dr. Carl Michalke. (Elektrotechnik in Einzeldarstellungen, Part IV. Edited by Dr. Gustav Benischke). Braunschweig, Friedrich Vieweg und Sohn, 1904.

Technische Hülfsmittel zur Beförderung und Lagerung von Sammelkörpern (Massengütern). Part II. By M. Buhle. Berlin, Julius Springer, 1904.

Die Wasserrohrkessel der Kriegs- und Handelsmarine. Ihre Bauart, Wirkungsweise, Behandlung und Bedienung. Von Walter Leps und Max Dietrich. Rostock, C. J. E. Volckmann, 1904.

Elemente des Wasserbaues. Für Studierende Höherer Lehranstalten und Jüngere Techniker. Von Eduard Sonne und Karl Esselborn. Leipzig, Wilhelm Engelmann, 1904.

Traité de Théorique et Pratique de la Résistance des Matériaux. Appliquée au Béton et au Ciment Armé. Par N. de Tedesco et A. Maurel. Paris, Libraire Polytechnique Ch. Béranger, 1904.

Central Station List and Manual of Electric Light. Published Quarterly the First Day of March, June, September and December. New York, McGraw Publishing Company, 1904.

The Municipal Year Book of the United Kingdom for 1904. Edited by Robert Donald. London, Edward Lloyd, Limited, 1904.

The Drainage of Town and Country Houses. A Practical Account of Modern Sanitary Arrangements and Fittings. By G. A. T. Middleton. London, B. T. Batsford, 1903.

Pollution of Tidal Waters, With Special Reference to Shellfish. Fourth Report of the Commissioners Appointed in 1898 to Inquire and Report What Methods of Treating and Disposing of Sewage may Properly be Adopted. London, Eyre and Spottiswoode, 1904.

Directory to the Iron and Steel Works of the United States. Embracing a Full Description of the Blast Furnaces, Rolling Mills, Steel Works, Tinplate and Terne Plate Works, and Forges and Bloomaries in the United States; also Classified Lists of the Wire Rod Mills, the Structural Mills, Plate Sheet, and Skelp Mills, Black Plate Mills, Rail Mills, Steel Casting Works, Bessemer Steel Works, Open Hearth Steel Works, and Crucible Steel Works. Compiled and Published by the American Iron and Steel Association. Sixteenth edition. Corrected to August 1, 1904. Philadelphia, American Iron and Steel Association, 1904.

Modern Practical Electricity. Electricity in the Service of Man; A Popular and Practical Treatise on the Applications of Electricity in Modern Life. Vol. IV. By R. Mullineux Walmsley. Chicago, W. T. Keener and Co.

Der Kaskadenumformer. Seine Theorie, Berechnung, Konstruktion und Arbeitweise. Von E. Arnold und J. L. la Cour. Sonderabdruck aus Sammlung elektrotechnischer Vorträge. Herausgegeben von Dr. Ernst Voit. Band VI. Berlin, Ferdinand Enke, 1904.

SUMMARY OF ACCESSIONS.

August 7th to September 12th, 1904.

Donations (including 14 duplicates and 4 numbers completing volumes of periodicals)	177
By purchase	13
Total	190

MEMBERSHIP.

ADDITIONS.

MEMBERS.

		Date of Membership.
BRACE, JAMES HENRY. Care, P., N. Y. & L. I. R. R., 20 West 34th St., New York City	Assoc. M. M.	Jan. 2, 1901 Sept. 6, 1904
HOLCOMBE, JOSEPH GALES. Prin. Asst. Engr., Bureau of Eng., Manila, Philippine Islands		May 4, 1904
ROBBINS, ALLAN APPLETON. Res. Engr., N. Y. & L. I. R. R., 1 East 42d St., New York City	Assoc. M. M.	Oct. 2, 1901 Sept. 6, 1904

ASSOCIATE MEMBERS.

ARMOUR, CHARLES WEBSTER L. 619 Garrison Ave., Fort Smith, Ark		June 1, 1904
BAKER, HUGH COSSART, Jr. 30 Gold St., New York City		May 4, 1904
FARNHAM, CHARLES HENRY. Care, Am. China Development Co., Canton, China		June 1, 1904
JOHNSTONE, WILLIAM BARD. 22 West 25th St., New York City.		May 4, 1904
WILD, HERBERT JOSEPH. Asst. to Supt. of Constr., U. S. Life Saving Service, 721 Broad St., Meriden, Conn	Jun. M.	Mar. 31, 1903 April 6, 1904

JUNIORS.

BANKER, EDWARD WARREN. 2d Lieut., U. S. M. C., U. S. Naval Academy, Annapolis, Md		Mar. 1, 1904
CRAWLEY, ERNEST WILLARD. 305 Exchange Bldg., New Haven, Conn		May 31, 1904

CHANGES OF ADDRESS.

MEMBERS.

BELKNAP, WILLIAM ETHELBERT ....Supt., Communipau Coal Co., Jersey City, N. J.; Address, 17 Battery Pl., New York City.

BELL, ALONZO CLARENCE ....920 Hibernia Bank and Trust Bldg., New Orleans, La.

BISBEE, FRED MILTON ....La Junta, Colo.

BIXBY, WILLIAM HERBERT.............Col., Corps of Engrs., U. S. A., Care, Military Secy., War Dept., Washington, D. C.

CAMPBELL, JOHN LOGAN ....Chf. Engr., Gasconade Ry. Constr. Co., Prin. Asst. Engr., St. L., K. C. & C. Ry., El Paso, Tex.

CORNELL, GEORGE BIRDSALL............20 Broad St., New York City.

CROSBY, BENJAMIN LINCOLN............Prin. Asst. Engr., N. P. Ry. Co., Tacoma, Wash.

CUSHING, EDWARD BENJAMIN .... Gen. Supt., M. L. & T. R. R., New Orleans, La.

DAVISON, GEORGE STEWART .... Care, J. M. Guffey Petroleum Co., Beaumont, Tex.

GOODALE, LOOMIS FARRINGTON .... Engr., M. of W. Missouri Dist., C., B. & Q. Ry. Co., 610 Globe Democrat Bldg., St. Louis, Mo.

GOULD, WILLIAM TILLOTSON .... Care, Penn., N. Y. & L. I. R. R. Co., 225 West 33d St., New York City.

HILLYER, WILLIAM ROSS .... Asst. Commr. of Public Works, Borough of Richmond, City of New York, Richmond Bldg., New Brighton (Res., 160 Richmond Ave., Port Richmond), N. Y.

JOHNSON, LORENZO MEDICI .... Pres., Pitts., Shawmut & Northern R. R. Co., 45 Wall St., New York City.

JOHNSON, LUCIEN SAMUEL .... Dam 11, Ohio River, Wellsburg, W. Va.

KINSLEY, THOMAS PEARSON .... Broadway and 187th St., New York City.

MINER, CHARLES AUGUSTINE .... 146 East 3d St., Oswego, N. Y.

MÜNSTER, ANDREW WENDELBO .... Acting Chf. Engr., Chic. Gt. West. Ry., St. Paul, Minn.

O'ROURKE, JOHN FRANCIS .... Cons. Engr., 1 West 34th St., New York City.

PASCHKE, THEODORE .... 1 West 100th St., New York City.

ROBINSON, ALBERT FOWLER .... Bridge Engr., A., T. & S. F. Ry. System, Room 1000, Ry. Exchange Bldg., Chicago (Res., 608 Marion St., Oak Park), Ill.

ROBINSON, GEORGE HENRY .... Drawer 821, Vancouver, B. C., Canada.

RUGGLES, WILLIAM BURROUGHS .... Care, Isthmian Canal Comm., Culebra, Canal Zone, Panama, Care, New York Post Office.

SMITH, CHARLES WILLIAM .... 393 Lewis Ave., Brooklyn, N. Y.

THOMAS, GEORGE EDWARD .... Superv. Engr., I. B. R. T., 59th St. Power House (Res., 50 West 92d St.), New York City.

WEBB, WALTER LORING .... Cons. Engr., 2222 Land Title Bldg., Philadelphia, Pa.

WEBSTER, CHARLES EDWARD .... Cons. Engr., South Bethlehem, Pa.

WILLIAMS, GARDNER STEWART .... 1503 Washtenaw Ave., Ann Arbor, Mich.

ASSOCIATE MEMBERS.

ADGATE, FREDERICK WHITNEY .... Supt. for The Foundation Co., Terre Haute, Ind.

BARNES, WILLIAM THOMAS .... 61 Elm St., Waterville, Me.

BASCOME, WESTERN RADFORD .... 3759 Westminster Pl., St. Louis, Mo.

BAUER, JACOB LOUIS .... 25 Dix Bldg., Elizabeth, N. J.

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DEATHS.

BERGENGREN, FRITZ CARL ANDERS GEORG. Elected Associate Member, December 2d, 1896; died August 8th, 1904.
BURTON, STANDISH BARRY............Elected Member June 1st, 1898; died August 13th, 1904.
GARDNER, GEORGE CLINTON...........Elected Member, November 3d, 1875; died August 12th, 1904.
GREENE, JOSEPH NORTON.............Elected Member, October 5th, 1887; died July 26th, 1904.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(August 4th to September 10th, 1904.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Engineer*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, $1.
(29) *Journal*, Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des École Spéciales de Gand*, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Économique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill., 10c.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.
(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 410 Penn Ave., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E. London, England.
(64) *Power*, New York City, 20c.
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(69) *Mill Owners*, New York City, 10c.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England
(72) *Street Railway Review*, Chicago, 30c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal, London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.

LIST OF ARTICLES.

Bridge

A Practical Method of Adjusting the Cables of Suspension-Bridges; with some Notes on Wire-Rope Cables, Strands, and Anchorages, from recent Australian Practice.* Ernest Macartney de Burgh, M. Inst. C. E. (63) Vol. 156.
A Light 510-Foot Suspension Bridge.* (14) Aug. 6.
A Skew-Span Double-Track 54-Foot Reinforced Concrete Arch.* (14) Aug. 6.
Sydney Harbour Bridge.* (12) Aug. 12.
Pile and Timber Trestle Bridges on the Santa Fe.* A. F. Robinson. (From *Bulletin* of the Amer. Ry. Eng. and Maintenance of Way Assoc.) (15) Aug. 12.
The Erection of the 282-Foot Triangular Span of the Fraser River Bridge. (14) Aug. 13.
New Design of Reinforcement for Concrete Steel Girders. E. A. S. Whitford. (46) Aug. 13.
The Division Street Bascule Bridge, Chicago. (14) Aug. 20.
The Merrimac River Bridge at Newburyport.* (14) Aug. 20.
Novel Bridge Construction. (62) Aug. 25.
Suspension Ferry Over the Loire River at Nantes, France.* B. H. Ridgely. (15) Aug. 26.
The Erection of the Tenth Street Bridge, Pittsburg.* (14) Serial beginning Aug. 27.
The New Main Span of the Plattsmouth Bridge.* (14) Aug. 27.
Transporter Bridges at Nantes, Bizerte and Rouen.* (10) Sept.
The Tornado of August 20, 1904, at St. Paul, Minn.* (Its effect on bridges.) C. A. P. Turner, M. Am. Soc. C. E. (13) Sept. 1.
American Bridge Building in Equatorial Africa.* (14) Serial beginning Sept. 3.
Progress in Railroad Bridge Building. F. C. McMath. (Paper read before the Detroit Eng. Soc.) (62) Sept. 8.
Reinforced Concrete Highway Bridges in New Jersey. (14) Sept. 10.
Les Nouveaux Grands Ponts sur l'East River à New York.* G. Richou. (33) Serial beginning July 30.
Les Ponts du Haut—Ogoôué dans le Congo Français.* Lt.-Colonel Gisclard. (33) Aug. 27.

Electrical.

The Storage Battery.* J. Lester Woodbridge. (8) July.
Direct-Reading Measuring Instruments for Switchboard Use.* Kenelm Edgcumbe, Assoc. M. Inst. E. E., and Franklin Punga. (77) July.
Power Station Design.* C. H. Merz, M. Inst. E. E., and Wm. McLellan, Assoc. M. Inst. E. E. (77) July.
The Steam Turbine as Applied to Electrical Engineering.* C. A. Parsons, G. G. Stoney, C. P. Martin. (77) July.
The Distribution of Electricity in Shipyards and Engine Works.* A. J. Anderson, Assoc. M. Inst. E. E. (77) July.
Electrical Engineering in South Africa. John Roberts. (73) July 29.
The Organisation and Management of a Central Station Meter Department.* A. J. Cridge. (Abstract of Paper read before the Municipal Elec. Convention.) (26) July 29.
Some Notes on the Bristol Electricity Works Fire.* H. Faraday Proctor. (Paper read before the Incorporated Mun. Elec. Assoc.) (73) July 29.
Synthetic Wireless Telegraphy.* A. Frederick Collins. (27) July 30.
Telephonic Tendencies. (27) July 30.
Winding a Direct-Current Generator Armature.* Arthur Wagner. (From *Electric Club Journal*.) (47) July 30.
Some of the Electrical Features of the Exposition.* E. B. Ellicott. (4) Aug.
The Works of the General Electric Company.* (11) Serial beginning Aug. 5.
Ipswich Electric Lighting and Tramways Undertaking.* (26) Aug. 5.
The Location of Lightning Arresters. (26) Aug. 5.
"Zone" Dynamos and Motors.* Henry F. Joel, Assoc. M. Inst. C. E. (26) Serial beginning Aug. 5.
The Adaptability of Electrical Driving. B. Longbottom. (Abstract of Paper read before the Manchester Assoc. of Engrs.) (26) Serial beginning Aug. 5.
Armature Reaction in Alternators.* James B. Henderson and John S. Nicholson. (73) Aug. 5.
The Power Plant of the Littleton Creamery, Denver. (14) Aug. 6.
The 70 000-Volt Continuous Current Transmission Experiments at Sécheron.* A. Steens. (14) Aug. 6.
A Large New Steam Turbine Power Plant Installation for Dubuque, Iowa.* (17) Aug. 6.
The Guanajuato, Mexico, Power Transmission.* Robert McF. Doble. (27) Serial beginning Aug. 6; Abstract (14) Aug. 13.
Method for Measuring the Output of Induction Motors.* E. Alexanderson. (27) Aug. 6.
New System of Wire Fixing and Protecting Apparatus for Overhead Electric Light and Power Wires and Cables.* (27) Aug. 6.
Single-Phase Series Motor.* Ralph McNeill. (27) Aug. 6.

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Electrical—(Continued).

Aluminium Electrical Conductors. Roderick J. Parke. (Abstract of Paper read before the Canadian Electrical Assoc.) (47) Aug. 6.
Leek Electricity Works.* (26) Aug. 12.
Recent Advances in Electro-Chemistry. Bertram Blount. (29) Serial beginning Aug. 12.
Electrotechnical Systems of Units.* David Robertson. (73) Aug. 12.
The Largest Generating Unit at the World's Fair.* (27) Aug. 13.
The Relation of Telephone Traffic to Efficient Service.* Howard S. Knowlton. (27) Aug. 13.
Rigidance, a New Name for the Reciprocal of Capacity.* James E. Ives. (27) Aug. 13.
Medium-Span Line Construction.* C. A. Copeland, Assoc. Am. Inst. E. E. (13) Aug. 18.
Electrical Methods of Measuring Temperature.* (62) Aug. 18.
Electric Tides and Earth Currents.* W. H. Sharp. (26) Serial beginning Aug. 19.
Messrs. Siemens Brothers & Co.'s New Manufactory at Stafford.* (26) Serial beginning Aug. 19.
Iron Losses in Dynamos. (26) Aug. 19.
Naval Telegraph Instruments: Electric Telegraphs. (26) Aug. 19.
Orling-Armstrong Electro-Capillary Recorder.* (26) Aug. 19.
A Method of Laying Bare Underground Mains.* (73) Aug. 19.
A 500 000-Volt Transformer at the St. Louis Exposition.* (27) Aug. 20.
A Practical Test on Commutation.* Arthur Keller. (27) Aug. 20.
A Unique Storage Battery Installation.* Henry Floy. (27) Aug. 20.
Economical Features of the Austin Telephone Company's All-Cable Plant, Austin, Minn.* (27) Aug. 20.
Will Mechanical Stokers under Steam Boilers Pay? S. D. Shook. (62) Aug. 25.
Voltage Regulation in Alternating Current Systems.* H. S. Meyer. (Paper read before the Liverpool Eng. Soc.) (62) Serial beginning Aug. 25.
A Natal Three-Phase Transmission Plant.* (26) Aug. 26.
On Large Bulb Incandescent Electric Lamps as Secondary Standards of Light.* J. A. Fleming. (Paper read before the British Assoc. for the Advancement of Science.) (26) Aug. 26; Abstract (73) Aug. 26.
Flame Temperatures in Internal Combustion Motors.* Dugald Clerk. (Paper read before the British Assoc. for the Advancement of Science.) (73) Serial beginning Aug. 26.
The Distribution of Magnetic Induction in Multipolar Armatures.* W. M. Thornton. (Paper read before the British Assoc. for the Advancement of Science.) (73) Aug. 26.
The Janus Telephone System.* Alfred Gradenwitz. (19) Aug. 27.
Multiple Control of Press Motors in San Francisco.* Wyatt H. Allen. (27) Aug. 27.
Two Methods of Locating Faults in Telephone Cables.* H. B. Stabler. (27) Aug. 27.
A Physical Conception of Some Alternating-Current Phenomena.* Lamar Lyndon. (27) Serial beginning Aug. 27.
Deflection of Stator of Direct-Current and Three-Phase Generators.* F. Niethammer. (27) Aug. 27.
General Electric Company's New Compensated Motor for Railway Service.* (13) Aug. 25; (15) Aug. 26; (40) Aug. 26; (14) Aug. 27; (17) Aug. 27; (18) Aug. 27; (27) Aug. 27.
Circuit-Breakers.* W. M. Scott. (3) Sept.
The Electric Equipment of Workshops and Factory Buildings. Percival Robert Moses. (9) Sept.
Vacuum-Tube Lighting.* D. McFarlan-Moore. (10) Sept.
The Broad Street Station of the Citizens Light, Heat and Power Company, Johnstown, Pa.* (64) Sept.
Operating Direct Current Generators.* (76) Sept.
The Hydro-Electric Station at Bogota.* A. Steens. (14) Sept. 3.
Rock Creek, Oregon, Power and Transmission System.* (27) Sept. 3.
Treating Incandescent Lamp Filaments.* Ralph McNeill. (27) Sept. 3.
Calculation of Alternator Regulation.* C. E. Canfield. (27) Sept. 3.
Steam Turbine-Driven Central Station (Johnstown, Pa.).* (27) Sept. 3.
Wireless Telegraph Service in St. Louis.* Cloyd Marshall. (27) Sept. 3.
Dispelling Fog by Electricity: An Experimental Proof.* A. Frederick Collins. (46) Sept. 3.
A 16-Mile, 80 000-Volt, Experimental Electric Transmission Line. A. C. Balch. (Paper read before the Pacific Coast Electric Transmission Assoc.) (13) Sept. 8.
The Power Plant of the Agnes Memorial Sanatorium.* Howard S. Knowlton. (14) Sept. 10.
Détermination des Conditions d'Économie Maxima pour le Fonctionnement des Lampes Électriques à Incandescence. Aymard Valéry. (36) July 25.
Soins a Donner aux Batteries d'Accumulateurs. E. Dieudonné. (36) Serial beginning July 25.
Cabestan Électrique à Courant Continu. Boy de la Tour. (34) Aug.
Les Cables Sous-Marins.* G. Gouraud. (36) Serial beginning Aug. 25.

Marine.

Boiler and Engine Test of a Small Steamboat. Warren Johnson. (1) July.
A Modern Ship and Engine Plant.* (41) Aug.

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The Draft of Vessels in Motion in Shallow Channels Compared with Their Stationary Draft. Henry N. Babcock, M. Am. Soc. C. E. (13) Aug. 4.
Report of the Committee on Naval Boilers (Great Britain). (11) Aug. 5.
Torpedo-Boat No. 298 for the French Navy.* (11) Aug. 5.
The Japanese Navy: On Recent Warships in the Japanese Navy.* Rear-Admiral Sa-Sow. (Paper read before the Japanese Inst. of Naval Archts.) (12) Aug. 12.
Practical Electrical Testing in the Navy. (26) Aug. 12.
Electricity and the New Steamer *Wolvin*.* Waldon Fawcett. (27) Aug. 18.
Speed Trial of the Battleship *Ohio*. (46) Aug. 13.
The United States Marine Corps.* Waldon Fawcett. (46) Aug. 18.
Naval Boilers. (47) Aug. 13.
Austrian Lloyd's Steam Navigation Company's T.-S. S. *Africa*.* (11) Serial beginning Aug. 19.
Admiral Togo's Flagship *Mikasa*. (46) Aug 20.
A New Apparatus for Coaling Warships at Sea or in Harbor. (19) Aug. 20.
The Fisheries Cruiser *Canada*.* (12) Aug. 26.
The Water-Tube Boilers of H. M. SS. *Medusa*, *Medea* and *Hermes*.* (11) Serial beginning Aug. 26.
Anti-Fouling Compositions (Paint for the Bottoms of Ships). (From *Farben Zeitung*.) (19) Aug. 27.
The Era of Iron-Clad Warfare.* J. P. Farley. (44) Serial beginning Sept.—Oct.
Some Notes on the Design and Building of the 21 000-Ton Steamships *Minnesota* and *Dakota*.* Charles R. Hanscom. (Abstract of Paper read before the Soc. of Naval Archts. and Marine Engrs.) (13) Sept. 1.
Launch of the Armored Cruiser *South Dakota*. (46) Sept. 3.
The *Minnesota*, the Largest Steamship ever Built in America.* (46) Sept. 3.
The Unarmored Composite Gunboat *Dubuque*.* (46) Sept. 10.
The New Cunard Turbine Steamers.* (46) Sept. 10.
La Navigation Sous-Marine. H. Noalhat. (36) Serial beginning Aug. 10.

Mechanical.

Notes on the Production and Thermal Treatment of Steel in Large Masses.* Cosmo Johns. (71) Vol. 65.
The Manufacture of Coke in the Hüssener Oven at the Clarence Iron Works, and its Value in the Blast-Furnaces.* C. Lowthian Bell. (71) Vol. 65.
The Design of the Steam Turbine: The Economic Value of the New Invention.* F. F. Coleman. (60) Apr.
The Economy of the Feed Water Heater.* O. S. Pedersen. (69) Apr.
Steam Traps: The Rational and Practical Development: The Special Design Founded on the Difference in the Co-Efficients of Brass and Iron.* John Platt. (69) Apr.
The Gas Machine in Factory Economy.* (69) Apr.
Commercial Gas Analysis with Stationary Hempel Apparatus. H. C. Babbitt. (58) May.
Pneumatic Stone Dressing Machines at the Wachusett Dam, Clinton, Mass.* (13) June 30.
The Use of Steel Scrap in the Foundry. W. G. Scott. (Abstract of Paper read before the Amer. Foundrymen's Assoc.) (13) June 30.
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Production and Transmission of Producer Gas. (47) July 30.
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The Growth of the Modern Machine Tool.* C. E. Coolidge. (From *Sibley Journal of Engineering*.) (41) Aug.
A New Chuck for Short Test Pieces.* T. D. Lynch. (Paper read before the Amer. Soc. for Testing Materials.) (41) Aug.
The Generation of Superheated Steam.* Otto Berner. (From *Zeitschrift* des Vereines Deutscher Ingenieure.) (64) Aug.
Specific Heat and Total Heat of Superheated Steam.* Geo. A. Orrok, Mem. A. S. M. E. (64) Aug.
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The Compounds of Vanadium in Brown-Coal Clay and the Green and Yellow Colors Which They Cause in Terra Cotta Products. (76) Serial beginning Aug. 1.
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An Efficient Ice Plant at East Hampton, N. Y.* (14) Aug. 6.
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Curtis Steam Turbine in the St. Louis Machinery Building.* (46) Aug. 13.
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Mechanical Draught for Boilers.* Walter Yates, M. I. Mech. E. (Paper read before the Inst. of Heat. and Vent. Engrs.) (47) Aug. 13; Abstract (22) July 29.
The Spontaneous Ignition of Coal. Alfred O. Doane. (13) Aug. 18.
The Porter Galvanizing Process.* (20) Aug. 18.
How to Calculate the Cylinder Bore, Stroke, etc. Th. L. Leon de Tissandier. (62) Aug. 18.
Steam-Turbine Construction.* O. Lasche. (11) Serial beginning Aug. 19.
Compressed Air and Pneumatic Tools in Engineering Works. Frank Fielden. (11) Aug. 19.
Tool-Grinding Machines at the St. Louis Exhibition.* (11) Aug. 19.
The Mumford Water-Tube Boiler.* (11) Aug. 19.
Mechanical Plant of the Hotel Lafayette, Buffalo.* (19) Aug. 20.
The Science of Burning Liquid Fuel.* W. N. Best. (19) Aug. 20.
Construction and Fencing of Hoists and Teagles.* W. Sydney Smith. (47) Serial beginning Aug. 20.
Westinghouse Steam Turbine.* (47) Aug. 20.
Station Meters.* Donald McDonald. (Read before the Congress of Gas Associations of America.) (24) Aug. 22.
The Engineer in the Foundry. R. Moldenke. (Paper read before the Amer. Foundry-men's Assoc.) (62) Serial beginning Aug. 25.
The Calorimetry of the Gases Exhausted from an Internal-Combustion Engine.* B. Hopkinson. (Abstract of Paper read before the British Assoc. for the Advancement of Science.) (11) Aug. 26.
The Manufacture of Bricks from Destructor Clinker.* (12) Aug. 26.
The Effects of Receiver Drop in a Compound Engine.* Joseph W. Hayward. (Paper read before the British Assoc. for the Advancement of Science.) (12) Aug. 26.
The Aeroplane. Rudolphe Sorean. (From *La Vie Automobile.*) (19) Aug. 27.
A Gasoline-Electric Automobile.* Emile Guarini. (19) Aug. 27.
Care and Management of Steam Boilers.* William H. Fowler, M. I. C. E. (47) Serial beginning Aug. 27.
Entropy in Steam Engine Practice.* (47) Serial beginning Aug. 27.
Engine Breakdowns and Their Lessons.* (47) Aug. 27.
Pressure Points. E. C. Jones. (Paper read before the Pacific Coast Gas Assoc.) (24) Aug. 29.
The Introduction of High-Speed Steel into a Factory. A. D. Wilt, Jr. (9) Sept.
Variable-Speed Appliances.* E. K. Hood. (10) Sept.
Drop Valve-Engines.* Charles Hurst. (10) Sept.
High-Speed Tool Steel, Its Manufacture and Use. J. M. Gledhill. (Abstract of Paper read before the Coventry Eng. Soc.) (21) Sept.
The New No. 3 Coke Plant of the Oliver & Snyder Steel Co.: A Description of the Layout and Equipment of a Complete Modern Plant * (45) Sept.
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Vol. XXX. SEPTEMBER, 1904. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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Vol. XXX. SEPTEMBER, 1904. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE STRUCTURAL DESIGN OF BUILDINGS.

By C. C. Schneider, M. Am. Soc. C. E.

To be Presented October 19th, 1904.

The object of this paper is to submit a set of specifications, for the structural work of buildings, for discussion and criticism.

As this subject has never been brought before this Society, it is expected that an exhaustive discussion will bring out some valuable suggestions from those who have had experience in building construction, and that this may result finally in a more uniform practice as well as in more uniformity in that portion of building ordinances relating to structural work.

These specifications were prepared originally for the instruction and guidance of the engineers employed in the various offices of the company with which the writer is connected. They were to be used not only in places where building laws do not exist, but also to supplement those local building laws which do not give sufficient data.

Since then the writer has made changes and revisions, which, in some instances, might be regarded as departures from the usual

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

practice, but, beyond this, it has been his aim to select what he considers the best practice of the present day.

These specifications are intended to cover only the structural features of buildings of the modern type, in which steel forms a part of the construction, such as would come naturally under the supervision of an engineer, and, therefore, they are not intended for the building trade, but for the use of educated engineers.

A proper and timely subject, to be included in specifications for structural work of buildings, is that of steel-concrete construction, of which a great deal has been used in later years in fire-proof buildings, etc. However, as this Society has recently appointed a committee to investigate and report on this subject, it was deemed advisable to omit steel-concrete construction from these specifications until the committee's investigations have given additional light on the subject.

The Appendix to this paper contains, in tabulated form, extracts from the various building laws which the writer has been able to obtain up to the present time. It will be noticed that the most striking feature in these building laws is their lack of uniformity as to the specified live load. The minimum live loads per square foot prescribed for floors of dwellings, hotels and apartment houses vary from 40 to 75 lb.; for floors of office buildings, from 60 to 150 lb.; for public assembly rooms, churches and theaters, from 80 to 150 lb.; for schools, from 75 to 150 lb., etc. To make the variation still greater, some building laws allow a reduction for columns and foundations on the permissible live loads specified for floors; others do not.

A similar variation also exists in the permissible unit strains allowed for different kinds of material.

In order that the specifications may be understood properly, it will be necessary to explain some of the more important clauses, and give the reasons which led the writer to adopt them.

LOADS.

Dead Load.—The dead load, or the weight of the structure itself, including permanent fixtures, can be ascertained easily by careful computation, and is a permanent and reliable quantity.

Live Load.—Attention has been called to the great difference in the live loads specified by the various building laws for buildings to be used for the same purpose, the differences being in some cases more than 100 per cent. These differences should be harmonized, and a more rational method of loading devised, which would produce a structure of ample strength, more particularly in its details and connections, without waste of material in places where it is not needed.

The writer's attention was first called by Theodore Cooper, M. Am. Soc. C. E., to the irrational practice of specifying a uniform live load per square foot; he thought the specified live loads should be a little more than mere guesswork. Since then the writer has been working in accordance with these suggestions, following the lines which have been recognized for years by engineers in specifying live loads for bridges.

The possible maximum superimposed or live loads on buildings for special purposes, such as warehouses or stores for particular kinds of goods, power-houses, department stores, etc., after their interior arrangement has been decided upon, can be accurately determined.

However, there are classes of buildings, the rooms of which may be occupied for various purposes at various times, such as office buildings, stores, hotels, apartment-houses, dwelling-houses, etc. Dwelling-houses are sometimes used for offices, and rooms in office buildings for light manufacturing purposes.

While it is impossible to foresee and provide for all possible contingencies, it is within the limits of possibility to provide for the varying conditions of loading which may occur in a building if used for the purpose for which it was intended.

Live Loads on Floors.—Mr. C. H. Blackall states*:

"The writer has repeatedly counted the number of persons in the various portions of theatres and music-halls, without once finding, even in crowded aisles and standing-room, an average of more than 40 or 50 lb. per sq. ft. extended over more than a few square feet."

This agrees also with the writer's observations.

A live load of 40 lb. per sq. ft., therefore, may be considered the maximum load to be provided for as a distributed load for all floors on which crowds of people may be expected to congregate, such as all kinds of rooms in dwelling-houses, apartment-houses, hotels, office buildings, schools, churches, theatres, concert halls, ballrooms, drill-rooms, etc.

* *American Architect*, August 26th, 1898.

Mr. Blackall, in conjunction with Mr. A. G. Everett, made a thorough investigation of the actual existing live loads of three office buildings in Boston. These loads were obtained by taking the actual weights of the furniture and contents and the greatest number of people known to be at any one time in an office, the average weight of one person being estimated at 150 lb. The greatest load was found in one of the offices of the Ames Building, amounting to 40.2 lb. per sq. ft.

In only 12.4% of the offices was the floor load in excess of 25 lb. per sq. ft., and in only 26% did it exceed 20 lb. per sq. ft. The greatest maximum average for all floors of any one of the three buildings was 17 lb. per sq. ft.

In accordance with these data, it may be considered safe to assume that a distributed live load of 40 lb. per sq. ft. will be sufficient to provide for a crowd of people as well as for the ordinary loads carried on floors used for offices or similar purposes.

The writer has investigated this subject and endeavored to discover extreme cases in order to find a method of concentrated loading to cover the same.

For this purpose, weights of all kinds of furniture were collected and their contents estimated. The weights were not taken as they actually existed, but as they would be if completely filled with the material for which they were intended.

It was found that the ordinary furniture, such as desks, tables, wardrobes, counters, chests. small safes, etc., may be discarded for extreme loads. The heaviest concentrated loads found in any office were safes.

The portable safes used in offices rarely ever weigh more than 5 000 lb. with contents. This load may be carried by one beam, and, as a safe of this weight is likely to be placed in any office, every floor joist should be calculated for a concentrated load of 5 000 lb. in any position.

The maximum weight of safes generally used in dwelling-houses is 2 000 lb.

The heaviest portable safe manufactured weighs 16 000 lb. and occupies a floor space of 69 by 45 in.

Safes of such excessive weight, however, are not placed on floors of office buildings in which no special provisions are made for them, unless arrangements are made to distribute the load over at least several beams.

In offices, the weights of all other furniture with contents do not approach that of safes.

Only a few cases of combinations of extreme loads were found to produce results similar to that of a concentrated load of 5 000 lb. They were as follows:

In a large room used as an engineering office, a number of cases with drawers holding drawings were placed in a double row, back to back, in the middle of the room, and used as a table on which to spread drawings. These cases were 31 in. wide and 36 in. high, weighing when completely filled 160 lb. per lin. ft., or both together 320 lb. per lin. ft.; but as their total width was 62 in., they may be considered as being carried by two beams.

A case of drawers for drawings, 31 by 44 in. and 5 ft. high, if completely filled, would weigh 1 200 lb. As there is a possibility of having a whole row of such cases placed along a partition, this would give a load of 326 lb. per lin. ft., which may extend the whole length of a beam.

The weight of a "Wernicke" bookcase about 6¼ ft. high was found to be 170 lb. per lin. ft. when completely filled with books.

A row of bookcases might be placed on each side of a partition for the whole length of the room, in which case the load would be 340 lb. per lin. ft. If the partition, instead of running parallel to the beams, should be placed at right angles to them, and if the beams are spaced at the usual distance of 5 or 6 ft. apart, the concentrated load would be only from 1 700 to 2 040 lb. on each beam.

These investigations appear to indicate that a concentrated load of 5 000 lb. on any point of a beam, and a uniform load of about 340 lb. per lin. ft. of beam, will probably cover all possibilities of extreme loading of floors used for office purposes.

A concentrated load of 5 000 lb. is equivalent to the following uniform loads per linear foot of beam of different spans:

Spans........................	10	20	30	40
Uniform load per linear foot....	1 000	500	333	250

If the span of the beam is 30 ft. and more, then the load of 340 lb. per lin. ft. would govern. However, as offices are rarely more than 30 ft. long, and as the probability of having an available continuous space of more than 30 ft. on each side of a partition fully occupied with bookcases, completely filled, is extremely small; and when it is considered that this load will not be carried entirely by one beam, the floor acting as a distributor, this may safely be neglected, and it may be assumed that a concentrated load of 5 000 lb. covers all ordinary contingencies.

In order to have a comparison between the system of concentrated loading and the uniform loads usually specified, Table 1 gives the equivalent loads per square foot for beams of different lengths and spacing.

TABLE 1.

Span of beam, in feet.	Distance between Centers of Beams, in Feet.			
	4	5	6	7
10	250	200	166	143
15	166	133	111	95
20	125	100	83	71
25	100	80	66	57
30	83	66	55	48
35	72	57	48	41
40	62	50	42	36

The application of a concentrated load to each beam has the additional advantage of having all beam connections proportioned for a load of 5 000 lb., and, therefore, not only obtains stronger connections and additional stiffness, but also makes provision for excessive concentrated loads during erection.

A concentrated load of 5 000 lb. applied to floor girders, that is, girders which carry beams, is not sufficient to cover all cases of extreme loading.

By laying out a great number of arrangements of office floors, with different spacings of columns and beams, and by applying different combinations of maximum loads, it was found that a uniform load of 1 000 lb. per lin. ft. of girder will generally cover all possible contingencies, unless the uniform load of 40 lb. per sq. ft. gives greater results.

The floor girders, therefore, have to be tried by three methods in order to ascertain which gives the greatest result:

First.—For a concentrated load of 5 000 lb.;

Second.—A uniform load of 1 000 lb. per lin. ft.;

Third.—A uniform load of 40 lb. per sq. ft. of floor area.

This method of loading, as specified for office buildings, will also provide ample safety if any rooms should be used for light manufacturing purposes.

By applying, in actual examples, the rules for floor loads of office buildings, as recommended by the writer, it is found, if the columns are spaced at 20 ft. between centers in either direction, and the beams at 5 ft. between centers, that the live load on the beams will be equivalent to 100 lb., and on the girders 50 lb., per sq. ft. of floor area.

If the columns were spaced at 25 ft. between centers, and the beams 5 ft., as before, the live load would be equal to 80 lb. per sq. ft. on the beams and 40 lb. per sq. ft. on the girders.

For buildings occupied as dwellings, a concentrated load of 2 000 lb. is recommended for beams and a uniform load of 500 lb. per lin. ft. for girders, in connection with a uniform floor load of 40 lb. per sq. ft.

As it has been demonstrated that a uniform load of 40 lb. per sq. ft. will scarcely ever be exceeded by a crowd of people, this load, with the excess loads specified for office buildings, will be sufficient for floors of schools, churches, theaters and places where seats are provided, but for places where strong vibrations may be expected, such as ballrooms, drillrooms, gymnasia, etc., 100% should be added to the uniform live load for impact and vibrations, and, in order to reduce the deflections, and consequently the vibrations, the depth of the beams and girders should be limited to one-fifteenth of their span.

Loads on Columns.

It has been the practice of many engineers and architects to allow smaller live loads on columns than those specified for the floor system, or to reduce the loads per square foot of floor area on columns from story to story downward toward the foundations. Rules to that effect are also incorporated in some building laws.

In the specifications proposed by the writer, the rules of the New York building laws have been adopted, *viz.*, to reduce the live load on columns in buildings more than five stories high 5% for each story (commencing with the columns carrying the second floor from the top), until a reduction of 50% is reached.

In order to provide for any possible excessive loads, and to keep the dimensions of the lighter columns within reasonable limits, it is specified that columns carrying floor loads shall be proportioned for a minimum live load of 20 000 lb., and the proportion of length divided by least radius of gyration of section shall be limited to 125.

Applying these rules to the columns of an office building gives, for the columns carrying the top floor, a live load of 40 lb. per sq. ft. of floor area (unless this load is exceeded by the concentrated load of 20 000 lb.), which is reduced to a minimum load of 20 lb. per sq. ft. This agrees very closely with the investigations of Messrs. Blackall and Everett, who found the average maximum live load on any one office floor to be 40.2 lb. per sq. ft., and the average total maximum for any one building to be 17 lb. per sq. ft.

Loads on Foundations.

Several failures, which have resulted from unequal settlement of foundations, have demonstrated that it is of the utmost importance that foundations should be proportioned properly, more particularly those not on solid rock, or where a settlement is to be expected. The reason for some of these failures was that the foundations were proportioned for the theoretical live loads, which never occurred. During construction, when the dead loads caused a settlement, those foundations which received the smallest amount of live load, therefore, were strained nearly to the limit of the permissible pressure, while those with a large amount of live load received a very much smaller pressure per square foot, thus causing unequal settlement.

For many years it has been the practice to reduce the live loads on foundations to less than the amounts allowed on the footing of columns, a provision which also exists in some building laws. However, as the average live load in a fire-proof building is only 20 lb. (or less) per sq. ft., and the dead load on the interior columns approximately 100 lb. per sq. ft., and on exterior columns considerably more; and, as the foundations have generally reached their maximum settlement before the building is occupied, it seems that the logically correct way to proportion foundations would be to consider the dead load only, but reduce the pressure per square foot so that the permissible pressure for combined dead and live load will not be exceeded. For example:

The base of that foundation which gets the greatest live load in proportion to the dead load has:

100 000 lb.	live load,
400 000 "	dead load,
500 000 "	total load.

The permissible pressure on the base of the foundation is 2 tons, or 4 000 lb. per sq. ft., and this foundation, therefore, would require an area of $\frac{500\ 000}{4\ 000} = 125$ sq. ft., which, if the live load be omitted, would give a pressure of 3 200 lb. per sq. ft. for dead load alone. Therefore, all foundations in this building should be proportioned so that the pressure from the dead load alone will not exceed 3 200 lb. per sq. ft.

Wind Loads.

All structures are exposed to high wind pressures occasionally, and there have been many disasters caused by structures having been blown down by the wind. All cases of this kind can be traced to inadequate provision in the design to resist these forces. It is not sufficient to compute the wind strains on the exposed surface of the finished building and depend upon the walls and partitions for bracing. The steel frame of a building is generally run up ahead of the walls and partitions. In several instances the framework of buildings has been wrecked during erection, or has been blown out of plumb and has had to be pulled back into place. Sometimes, temporary wooden braces or temporary adjustable rods have been used to hold the framework in line during erection. In one case the framework was so flimsy and shaky that the erectors were afraid to work on it, and, in order to make it safe during erection, tied it together with wire ropes. Certainly, this was not good practice. The steel frame of a building should be treated as an independent structure, the same as the towers of a viaduct, and should be able to resist the wind forces on all surfaces exposed during erection. This should be accomplished by substantial bracing, or by designing the columns and connections so that they may be able to resist the bending strains produced by wind pressure. No temporary makeshifts should be allowed. This method has the advantage of imparting additional stiffness to the framework.

In proportioning the members of the structure for these temporary wind strains, it is permissible to allow a higher unit strain than for permanent work, say 20 000 lb. per sq. in., or about two-thirds of the elastic limit.

Unit Strains.

The permissible pressure allowed on foundations on different kinds of soil, on concrete, masonry, brickwork, etc., have been compiled from different sources.

The permissible unit strains on steel are specified as 16 000 lb. per sq. in., which is approximately one-half of the elastic limit; therefore, giving a factor of safety of 2. This is in accordance with the best practice now in vogue for bridge work.

Material and Workmanship.

It was not deemed necessary to include in these specifications the quality of the ordinary building material, such as cement, concrete, stone masonry, brickwork, etc., as most of that material is of a local character and is generally well covered by architects' and engineers' specifications.

Cast Iron.—This is practically ruled out in these specifications, as it is the poorest of all metals used for structural purposes to resist bending and tension. It has been the cause of several disasters, and, in bridge work, has been entirely abandoned for many years. The use of cast iron in columns with the usual beam connections is to be particularly condemned, as the beams are supported by lugs or brackets cast on the columns, thus producing eccentric loading and bending strains.

Rolled Steel.—This material, of the grade called "structural steel," adopted by the American Railway Engineering and Maintenance-of-Way Association for bridge material, is specified for all structural parts, as it is considered the most reliable for structural purposes. It is moreover, a commercial article which can be purchased from any reputable manufacturer without extra cost.

The specifications for material and workmanship are practically the same as those adopted by the American Railway Engineering and Maintenance-of-Way Association, as far as they were applicable to structural work for buildings.

These specifications are divided into two parts:

Part I.—This contains the information necessary for computation and designing, such as loads, unit strains and details of construction.

Part II.—This covers the quality of material, the workmanship and the inspection.

This division is made so that each part may be used separately: Part I in the office, by the designer, and Part II in the shop, by the manufacturer and inspector, and, for this reason, the paragraphs in each part are numbered separately.

GENERAL SPECIFICATIONS FOR STRUCTURAL STEELWORK OF BUILDINGS.

PART I.—DESIGN.

Loads.

1.—Dead Load.—The "dead" load in all structures shall consist of the weight of walls, floors, partitions, roofs and all other permanent construction and fixtures.

2.—In calculating the "dead" loads, the weights of the different materials shall be assumed as given in Table 16.

3.—Live Load on Floors.—Table 2 gives the "live" load on floors, to be assumed for different classes of buildings. These loads consist of:

a.—A uniform load per square foot of floor area;

b.—A concentrated load which shall be applied to all points of the floor;

c.—A uniform load per linear foot for girders.

The maximum result is to be used in calculations.

The specified concentrated loads shall also apply to the floor construction between the beams for a length of 5 ft.

TABLE 2.—Live Loads.

Classes of buildings.	Live Loads, in Pounds.		
	Distributed load.	Concentrated load.	Load per linear foot of girder.
Dwellings, hotels and apartment-houses..	40	2 000	500
Office buildings..........................	40	5 000	1 000
Assembly rooms with fixed seats, like theaters, churches, schools, etc..........	40	5 000	1 000
Assembly rooms, without fixed seats, like ballrooms, gymnasia, armories, etc.....	80	5 000	1 000
Stables and carriage houses...............	70	5 000	1 000
Ordinary stores and light manufacturing.	40	8 000	1 000
Sidewalks in front of buildings............	100	10 000	......
Warehouses and factories.................	from 120 up	Special.	Special.
Charging floors for foundries.............	" 300 "	"	"
Power-houses, for uncovered floors........	" 200 "	The actual weights of engines, boilers, stacks, etc., shall be used, but in no case less than 200 lb. per sq. ft.	

4.—If heavy concentrations, like safes, armatures, or special machinery, are likely to occur on floors, provision should be made for them.

5.—Crane Loads and Impact.—For structures carrying traveling machinery, such as cranes, conveyors, etc., 25% shall be added to the strains resulting from such live load, to provide for the effects of impact and vibrations. (For crane loads, see Table 17.)

6.—Loads on Flat Roofs.—Flat roofs of office buildings, hotels, apartment-houses, etc., which are likely to be loaded by crowds of people, shall be treated as floors, and the same live loads shall be used as specified for hotels and dwelling-houses.

7.—Loads on Ordinary Roofs.—Ordinary roofs shall be proportioned to carry the following loads per square foot of exposed surface, applied vertically, to provide for dead and live loads combined:

Gravel or composition roofing.	On boards, flat pitch, 3 to 12 in., or less.....	45 lb.
	On boards, steep pitch, more than 3 to 12 in.	40 "
	On 3-in. flat tile or cinder concrete..........	55 "
Corrugated sheeting, or boards or purlins......		50 "
Slate.	On boards or purlins.......................	50 "
	On 3-in. flat tile or cinder concrete..........	65 "
Tile on steel purlins.......................................		55 "

For roofs in climates where no snow is likely to occur, reduce the foregoing total loads by 10 lb. per sq. ft.

8.—Large Roofs.—Large roofs, such as train-sheds, armories, public halls, etc., shall be proportioned to carry, in addition to their own weight:

A live load, representing snow, per horizontal square foot of roof of:

15 lb. for all slopes not exceeding 35°;

10 lb. for all slopes between 35 and 45 degrees.

The possibility of a partial snow loading has to be considered. The snow load can be neglected in certain climates, also in roofs having slopes exceeding 45°, if there are no snow guards or other obstructions.

9.—Loads on Columns.—For columns, the specified uniform live loads per square foot shall be used, with a minimum of 20 000 lb. per column.

10.—Reduction of Live Load on Columns.—For building more than five stories in height, these live loads may be reduced as follows:

For roof and top floor, no reduction;

For each succeeding lower floor, a reduction of 5% until 50% is reached, which is to be used for the columns of all remaining floors, *viz.*, the reduced load is to be used for the total floor area carried by the column.

11.—Loads on Foundations.—The live loads on foundations shall be assumed to be the same as for the footings of columns. The areas of the bases of the foundations shall be proportioned for the dead load only. That foundation which receives the largest ratio of live to dead load shall be selected and proportioned for the combined dead and live loads. The dead load on this foundation shall be divided by the area thus found, and this reduced pressure per square foot shall be the permissible working pressure to be used for the dead load of all foundations.

12.—Wind Pressure.—The wind pressure shall be assumed at 30 lb. per sq. ft. acting in either direction horizontally:

First.—On the sides and ends of buildings and on the actually exposed surface, or the vertical projection of roofs;

Second.—On the total exposed surfaces of all parts composing the metal framework. The framework shall be considered an independent structure, without walls, partitions or floors.

UNIT STRAINS.

Substructure.

13.—Foundations.—Permissible pressure on foundations, in tons per square foot:

Soft clay and wet sand	1
Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm, coarse sand	4
Firm, coarse sand and gravel	6

14.—Masonry.—Permissible working pressure in masonry, in tons per square foot:

Common brick, lime mortar	7
" " Rosendale cement mortar	8
" " Portland cement mortar	10
Hand-burned brick, Portland cement mortar	12
Rubble masonry, lime mortar	5
" " Rosendale cement mortar	6
" " Portland cement mortar	8
Coursed rubble, Portland cement mortar	10

Concrete for walls:

Rosendale	cement,	1-2-5	8
"	"	1-2-4	9
Portland	"	1-2-5	15
"	"	1-2-4	16

15.—Pressure on Wall-Plates.—The pressure of beams, girders, wall-plates, column bases, etc., on masonry shall not exceed the following, in pounds per square inch:

On brickwork with cement mortar	150
" rubble masonry with cement mortar	150
" Portland cement concrete	250
" first-class masonry, sandstone	200 to 300
" " " " limestone	300 to 500
" " " " granite	400 to 800

16.—Bearing Power of Piles.—The maximum load carried by any pile shall not exceed 40 000 lb. Piles driven in loose, wet soil shall not be strained to more than 850 lb. per sq. in. of their average cross-section.

The safe load on wooden piles shall be determined by the following formula:*

$$P = \frac{2\ W H}{s + 1}.$$

Where P = safe load on pile, in tons;

H = distance of free fall of hammer, in feet;

s = penetration of the pile for the last blow, in inches.

Superstructure.

Steel.

17.—Permissible Strains.—All parts of the structure shall be proportioned so that the sum of the dead and live loads, together with the impact, if any, shall not cause the strains to exceed those given in the following table:

	Pounds per square inch.
Tension, net section	16 000
Direct compression	16 000
Shear, on rivets and pins	12 000
Shear, on bolts	8 000
Shear, on plate-girder web (gross section)	10 000
Bearing pressure, on pins and rivets	24 000
Bearing pressure, on bolts	16 000
Fiber strain, on pins	24 000

*Engineering News formula.

18.—For wind bracing, and the combined strains due to wind and the other loading, the permissible working strains may be increased 25%, or to 20 000 lb. for direct compression or tension.

19.—Permissible Compression Strains.—For compression members, these permissible strains of 16 000 and 20 000 lb. per sq. in. shall be reduced by the following formula:

$$p = 16\,000 - 70\,\frac{l}{r}$$

$$p = 20\,000 - 90\,\frac{l}{r}$$

Where p = permissible working strain per square inch in compression;

l = length of piece, in inches, from center to center of connections;

r = least radius of gyration of the section, in inches.

20.—Provision for Eccentric Loading.—In proportioning columns, provision must be made for eccentric loading.

21.—Expansion Rollers.—The pressure per linear foot on expansion rollers shall not exceed 600 d, where d = diameter of rollers, in inches.

22.—Transverse Loading of Tension or Compression Members.—When a floor, wall or other weight rests directly on the chord of a truss, said chord shall be proportioned so that the sum of the strains per square inch on the outer fiber, resulting from direct compression or tension, and three-fourths of the maximum bending moment (the chord being considered as a beam of one panel length, supported at the ends) shall not exceed the specified limiting strains in tension or compression, the proper amount of impact, if any, being added to each kind of loading.

23.—The bending moments at panel points shall be assumed equal to that in the center, but in opposite direction.

24.—Combined Strains.—All other members which are subject to direct strain, in addition to bending moment, shall be calculated in a similar manner.

25.—Alternate Strains.—Members and connections subject to alternate strains shall be proportioned for the strain giving the largest section.

26.—Net Sections.—Net sections must be used in calculating tension members, and, in deducting rivet holes, they must be taken ⅛ in. larger than the nominal size of the rivets.

27.—Pin-connected riveted tension members shall have a net section through the pin holes 25% in excess of the net section of the body of the member. The net section back of the pin hole shall be at least 0.75 of the net section through the pin hole.

28.—Compression Members Limiting Length.—No compression member shall have a length exceeding 125 times its least radius of gyration, except those for wind and lateral bracing, which may have a length not exceeding 150 times the least radius of gyration.

29.—Plate Girders.—Plate girders shall be proportioned on the assumption that one-eighth of the gross area of the web is available as flange area. The compression flange shall have the same sectional area as the tension flange, but the unsupported length of the flange shall not exceed 30 times its width.

30.—In plate girders used as crane runways, the unsupported length of the compression flange shall not exceed 20 times its width.

31.—Web Stiffeners.—The web shall have stiffeners at the ends and inner edges of bearing plates, and at all points of concentrated loads, and also at intermediate points, when the thickness of the web is less than one-sixtieth of the unsupported distance between flange angles, generally not farther apart than the depth of the full web plate, with a minimum limit of 5 ft.

32.—Rolled Beams.—I-beams, and channels used as beams or girders, shall be proportioned by their moments of inertia.

33.—Limiting Depth of Beams and Girders.—The depth of rolled beams in floors shall be not less than one-twentieth of the span, and if used as roof purlins, not less than one-thirtieth of the span.

In case of floors subject to shocks and vibrations, the depth of beams and girders shall be limited to one-fifteenth of the span.

34.—Field Connections.—In field connections, the number of rivets shall be increased 15 per cent.

Cast Iron.

35.—Compression on Cast Iron.—The direct compression on cast iron shall not exceed 12 000 lb. per sq. in.

Timber.

36.—Timber.—The timber parts of the structure shall be proportioned in accordance with the unit strains in Table 3.

TABLE 3.—UNIT STRAINS, IN POUNDS PER SQUARE INCH.

Kind of timber.	Transverse loading.	End bearing.	Columns under 12 diameters.	Bearing across fiber.	Shear along fiber.
White Oak..................	1 300	1 300	1 000	500	200
Long-Leaf Pine.............	1 500	1 500	1 000	350	100
White Pine and Spruce.....	1 000	1 000	700	200	100
Hemlock....................	800	800	650	200	100

37.—Columns, the length of which exceeds twelve times their least diameter, shall be proportioned by the following formula:

$$p = \frac{C}{1 + \frac{l^2}{1\,000\,d^2}}$$

Where C = unit strains, as given in Table 3, for short columns;

c = length of column, in inches;

d = least side of column, in inches.

38.—Planking.—For thickness of floor and roof planking, see Table 18.

DETAILS OF CONSTRUCTION.

39.—Minimum Thickness of Material.—No steel less than ¼ in. thick shall be used, except for lining or filling vacant spaces.

40.—Adjustable Members.—Adjustable members in any part of structures shall preferably be avoided.

41.—Symmetrical Sections.—Sections shall preferably be made symmetrical.

42.—Connections.—The strength of connections shall be sufficient to develop the full strength of the member.

43.—No connection, except lattice bars, shall have less than two rivets.

44.—Floor Beams.—Floor beams shall generally be rolled steel beams, and the ends shall be attached to the webs of the floor girders with angle connections.

45.—For fire-proof floors, they shall be arranged, as to spacing and length, so that the dead and live loads together shall not cause a greater deflection of the beams than $\frac{1}{30}$ in. per foot of span. They shall generally be tied together with tie-rods at intervals not exceeding eight times the depth of the beams. Holes for tie-rods, where the construction of the floor permits, shall be spaced about 3 in. above the bottom of the beam.

46.—Beam Girder.—When more than one rolled beam is used to form a girder, they shall be connected by bolts and separators at intervals of not more than 5 ft. All beams having a depth of 12 in. and more shall have at least two bolts to each separator.

47.—Wall Ends of Beams and Girders.—Wall ends of a sufficient number of joists and girders shall be anchored securely, to impart rigidity to the structure.

48.—Wall-Plates and Column Bases.—Wall-plates and column bases shall be constructed so that the load will be well distributed over the entire bearings. If they do not get the full bearing on the masonry, the deficiency shall be made good with rust cement or Portland cement mortar.

49.—Floor Girders.—The floor girders may be rolled beams or plate girders; they shall preferably be riveted or bolted to columns by means of connection angles. Shelf angles or other supports may be provided for convenience during erection.

50.—Flange Plates.—The flange plates of all girders shall be limited in width, so as not to extend, beyond the outer line of rivets connecting them to the angles, more than 6 in., or more than eight times the thickness of the thinnest plates.

51.—Web Stiffeners.—Web stiffeners shall be in pairs, and shall have a close bearing against the flange angles. Those over the end bearing, or forming the connection between girder and column, shall be on fillers. Intermediate stiffeners may be on fillers or crimped over the flange angles. The rivet pitch in stiffeners shall not be more than 5 in.

52.—Web Splices.—Web plates of girders must be spliced at all points by a plate on each side of the web, capable of transmitting the full strain through splice rivets.

53.—Columns.—Columns shall be designed so as to provide for effective connections of floor beams, girders or brackets.

They shall preferably be continuous over several stories.

54.—Column Splices.—The splices shall be strong enough to resist the bending strain and make the columns practically continuous for their whole length.

55.—Trusses.—Trusses shall preferably be riveted structures, Heavy trusses, of long span, where the riveted field connection would become unwieldy, or for other good reasons, may be designed as pin-connected structures.

56.—Symmetrical Sections.—Main members of trusses shall be designed so that the neutral axes of intersecting members shall meet in a common point.

57.—Roof Trusses.—Roof trusses shall be braced in pairs in the plane of the chords.

Purlins shall be made of shapes, or riveted-up plate, or lattice girders.

Trussed purlins will not be allowed.

58.—Eye-Bars.—The eye-bars in pin-connected trusses composing a member shall be as nearly parallel to the axis of the truss as possible.

59.—Spacing of Rivets.—The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for ⅞-in. rivets, 2½ in. for ¾-in. rivets, 2¼ in. for ⅝-in. rivets and 1¾ in. for ⅝-in. rivets.

60.—For angles with two gauge lines, the maximum shall be twice as great as given in Section 59 in each line with rivets staggered; and, where two or more plates are used in contact, rivets not more than 12 in. apart in any direction shall be used to hold the plates together.

61.—The pitch of the rivet, in the direction of the strain, shall not exceed 6 in., nor 16 times the thinnest outside plate connected, and not more than 50 times that thickness at right angles to the strain.

62 —Edge Distance.—The minimum distance from the center of any rivet hole to a sheared edge shall be 1½ in. for ⅞-in. rivets, 1¼ in. for ¾-in. rivets, 1⅛ in. for ⅝-in. rivets and 1 in. for ½-in. rivets, and to the rolled edge, 1¼, 1⅛, 1 and ⅞ in., respectively.

63.—The maximum distance from any edge shall be eight times the thickness of the plate.

64.—Maximum Diameter.—The diameter of the rivets in any angle carrying calculated strains shall not exceed one-quarter of the width of the leg in which they are driven. In minor parts, rivets may be ⅛ in. greater in diameter.

65.—Pitch at Ends.—The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets for a length equal to one and one-half times the maximum width of the member.

66.—Tie-Plates.—The open sides of compression members shall be provided with lattice having tie-plates at each end and at intermediate points where the lattice is interrupted. The tie-plates shall be as near the ends as practicable. In main members, carrying calculated strains, the end tie-plates shall have a length not less than the distance between the lines of rivets connecting them to the flanges, and intermediate ones not less than half this distance.

Their thickness shall be not less than one-sixtieth of the same distance.

67.—Lattice.—The thickness of lattice bars shall be not less than one-fortieth for single lattice and one-sixtieth for double lattice, of the distance between end rivets; their width shall be in accordance with the following:

For 15-in. channels, or built sections with 3½ and 4-in. angles..................	2½ in. (⅞-in. rivets);
For 12, 10 and 9-in. channels, or built sections with 3-in. angles............	2¼ in. (¾-in. rivets);
For 8 and 7-in. channels, or built sections with 2½-in. angles.............	2 in. (⅝-in. rivets);
For 6 and 5-in. channels, or built sections with 2-in. angles...............	1¾ in. (½-in. rivets).

68.—Lattice bars with two rivets shall generally be used in flanges more than 5 in. wide.

69.—Angle of Lattice.—The inclination of lattice bars with the axis of the member, generally, shall be not less than 45°, and when the distance between the rivet lines in the flanges is more than 15 in., if a single rivet bar is used, the lattice shall be double and riveted at the intersection.

70.—Spacing of Lattice.—The pitch of lattice connections, along the flange divided by the radius of gyration of the flange angle about an axis, through its center of gravity, perpendicular to the plane of the lattice, shall be less than the corresponding ratio of the member as a whole.

71.—Faced Joints.—Abutting joints in compression members when faced for bearing shall be spliced sufficiently to hold the connecting members accurately in place.

72.—All other joints in riveted work, whether in tension or compression, shall be fully spliced.

73.—Pin Plates.—Pin holes shall be reinforced by plates where necessary; and at least one plate shall be as wide as the flange will allow. Where angles are used, the plates shall be on the same side as the angles; they shall contain sufficient rivets to distribute their portion of the pin pressure to the full cross-section of the member.

74.—Pins.—Pins shall be long enough to insure a full bearing of all parts connected upon the turned-down body of the pin.

75.—Members packed on pins shall be held against lateral movement.

76.—Bolts.—Where members are connected by bolts, the body of these bolts shall be long enough to extend through the metal. A washer at least $\frac{3}{16}$ in. thick shall be under the nut.

77.—Fillers.—Fillers between parts carrying strain shall have a sufficient number of independent rivets to transmit the strain to the member to which the filler is attached.

78.—Temperature.—Provision shall be made for expansion and contraction, corresponding to a variation of temperature of 150° fahr., where necessary.

79.—Rollers.—Expansion rollers shall be not less than 4 in. in diameter.

80.—Stone Bolts.—Stone bolts shall extend not less than 4 in. into granite pedestals and 8 in. into other material.

81.—Anchorage.—Columns which are strained in tension at their base shall be anchored to the foundations.

82.—Anchor bolts shall be long enough to engage a mass of masonry, the weight of which shall be one and one-half times the tensile strain in the anchor.

83.—Bracing.—Lateral, longitudinal and transverse bracing in all structures shall be preferably composed of rigid members.

PART II.—MATERIAL AND WORKMANSHIP.

Material.

1.—Steel.—All parts of the structures shall be of rolled steel, except column bases, bearing plates or minor details, which may be of cast iron or cast steel. No cast iron shall be used in pieces which will have to resist tension or bending strains.

2.—Process of Manufacture.—Steel may be made by the open-hearth or by the Bessemer process.

3.—The chemical and physical properties shall conform to the limits given in Table 4.

TABLE 4.—Schedule of Requirements.

Chemical and physical properties.	Structural steel.	Rivet steel.	Steel castings.
Phosphorus, maximum.. Basic..	0.04 per cent.	0.04 per cent.	0.05 per cent.
Phosphorus, maximum.. Acid...	0.08 per cent.	0.04 per cent.	0.08 per cent.
Sulphur, maximum...............	0.05 per cent.	0.04 per cent.	0.05 per cent.
Ultimate tensile strength, Pounds per square inch..........	Desired 60 000	Desired 50 000	Not less than 65 000
Elongation: minimum percentage in 8 in..................	1 500 000* / Ult. tensile str'gth	1 500 000 / Ult. tensile str'gth	....
Elongation: minimum percentage in 2 in....................	22	....	18
Character of fracture............	Silky	Silky	Silky or fine granular.
Cold bends without fracture.....	180° flat.†	180° flat§	90

* See Paragraph 12. † See Paragraphs 14 and 15. § See Paragraph 16.

4.—The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

5.—Allowable Variations.—Tensile tests of steel showing an ultimate strength within 5 000 lb. of that desired will be considered satisfactory.

6.—Chemical Analyses.—Chemical determinations of the percentages of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector.

7.—Form of Specimens for Plates, Shapes and Bars.—Specimens for tensile and bending tests, for plates, shapes and bars, shall be made by cutting coupons from the finished product, which shall have both faces rolled and both edges milled to the form shown by Fig 1; or with both edges parallel; or they may be turned to a diameter of ¾ in. for a length of at least 9 in., with enlarged ends.

8.—Rivets.—Rivet rods shall be tested as rolled.

Pins and Rollers.—Specimens shall be cut from the finished rolled or forged bar, in such manner that the center of the specimen shall be 1 in. from the surface of the bar. The specimen for the tensile test shall be turned to the form shown by Fig. 2. The specimen for the bending test shall be 1 in. by ½ in. in section.

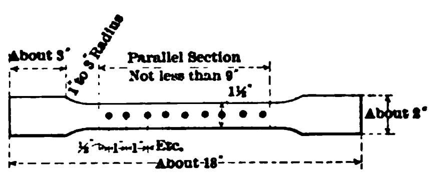

FIG. 1.

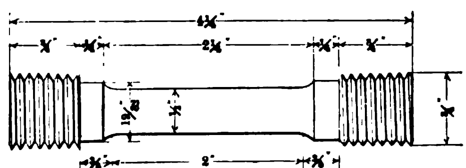
FIG. 2.

9.—Steel Castings.—The number of tests will depend on the character and importance of the castings. Specimens shall be cut cold from coupons moulded and cast on some portion of one or more castings from each melt, or from the sink-heads, if the heads are of sufficient size. The coupon or sink-head, so used, shall be annealed with the casting before it is cut off. Test specimens shall be of the form prescribed for pins and rollers.

10.—Annealed Specimens.—Material which is to be used without annealing or further treatment shall be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen for tensile tests representing such material shall be cut from properly annealed or similarly treated short lengths of the full section of the bar.

11.—Number of Tests.—At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing ⅜ in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

12.—*Modifications in Elongation.*—For material less than $\frac{5}{16}$ in. and more than $\frac{3}{4}$ in. in thickness, the following modifications will be allowed in the requirements for elongation:

a.—For each $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in., a deduction of $2\frac{1}{2}\%$ will be allowed from the specified elongation.

b.—For each $\frac{1}{8}$ in. in thickness above $\frac{3}{4}$ in., a deduction of 1% will be allowed from the specified elongation.

c.—For pins and rollers more than 3 in. in diameter, the elongation in 8 in. may be 5% less than that specified in Paragraph 3.

13.—*Bending Tests.*—Bending tests may be made by pressure or by blows. Plates, shapes and bars less than 1 in. thick shall bend as called for in Paragraph 8.

14.—*Thick Material.*—Full-sized material, for eye-bars and other steel 1 in. or more in thickness, tested or rolled, shall bend cold 180° around a pin the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of the bend.

15.—*Bending Angles.*—Angles $\frac{3}{4}$ in. and less in thickness shall open flat, and angles $\frac{1}{2}$ in. and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by the inspector.

16.—*Nicked Bends.*—Rivet steel, when nicked and bent around a bar of the same diameter as the rivet rod, shall give a gradual break and a fine, silky uniform fracture.

17.—*Finish.*—Finished material shall be free from injurious seams, flaws, cracks, defective edges, or other defects, and shall have a smooth, uniform, workmanlike finish. Plates 36 in. and less in width shall have rolled edges.

18.—*Stamping.*—Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Steel for pins and rollers shall be stamped on the end. Rivet and lattice steel and other small parts may be bundled with the above marks on an attached tag.

19.—*Defective Material.*—Material which, subsequent to the foregoing tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, will be rejected at the shop, and shall be replaced by the manufacturer at his own cost.

20.—*Allowable Variation in Weight.*—A variation in cross-section or weight of each piece of steel of more than $2\frac{1}{2}\%$ from that specified will be sufficient cause for rejection.

21.—Cast Iron.—Iron castings shall be made of tough, gray iron, free from injurious cold-shuts or blow-holes, true to pattern and of workmanlike finish. Test pieces 1 in. square shall be capable of sustaining on a clear span of 12 in. a central load of 2 500 lb. or more, and deflect at least 0.15 in. before rupture.

Workmanship.

22.—General.—All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works.

23.—Straightening Material.—Material shall be thoroughly straightened in the shop, by methods which will not injure it, before being laid off or worked in any way.

24.—Finish.—Shearing shall be done neatly and accurately, and all portions of the work exposed to view shall be finished neatly.

25.—Rivets.—The size of rivets called for on the plans shall be understood to mean the actual size of the cold rivet before heating.

26.—Rivet Holes.—The diameter of the punch for material not more than $\frac{5}{8}$ in. thick shall be not more than $\frac{1}{16}$ in., nor that of the die more than $\frac{1}{8}$ in., larger than the diameter of the rivet. Material more than $\frac{5}{8}$ in. thick, except minor details, shall be sub-punched and reamed or drilled from the solid.

27.—Punching.—Punching shall be done accurately. Slight inaccuracy in the matching of holes may be corrected with reamers. Drifting to enlarge unfair holes will not be allowed. Poor matching of holes will be cause for rejection, at the option of the inspector.

28.—Assembling.—Riveted members shall have all parts well pinned up and firmly drawn together with bolts before riveting is commenced. Contact surfaces shall be painted. (See Paragraph 52.)

29.—Lattice Bars.—Lattice bars shall have neatly rounded ends, unless otherwise called for.

30.—Web Stiffeners.—Stiffeners shall fit neatly between the flanges of girders. Where tight fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

31.—Splice Plates and Fillers.—Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ in. of flange angles.

32.—Connection Angles.—Connection angles for floor girders shall be flush with each other and correct as to position and length of girder.

33.—Riveting.—Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

34.—Rivets shall look neat and finished, with heads of approved shape, full, and of equal size. They shall be central on the shank and shall grip the assembled pieces firmly. Recupping and caulking will not be allowed. Loose, burned, or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjoining metal. If necessary, they shall be drilled out.

35.—Turned Bolts.—Wherever bolts are used in place of rivets which transmit shear, such bolts must have a driving fit. A washer not less than $\frac{1}{4}$ in. thick shall be used under the nut.

36.—Members to be Straight.—The several pieces forming one built member shall be straight and shall fit closely together, and finished members shall be free from twists, bends or open joints.

37.—Finish of Joints.—Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view. In compression joints depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

38.—Eye-Bars.—Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of the heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body with a silky fracture, when tested to rupture. The thickness of the head and neck shall not vary more than $\frac{1}{16}$ in. from the thickness of the bar.

39.—Boring Eye-Bars.—Before boring, each eye-bar shall be perfectly annealed and carefully straightened. Pin holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ in. smaller in diameter than the pin holes can be passed through the holes at both ends of the bars at the same time.

40.—Pin Holes.—Pin holes shall be bored true to gauges, smooth and straight; at right angles to the axis of the member, and parallel to each other, unless otherwise called for. Wherever possible, the boring shall be done after the member is riveted up.

41.—The distance from center to center of pin holes shall be correct within $\frac{1}{32}$ in., and the diameter of the hole not more than $\frac{1}{50}$ in. larger than that of the pin, for pins up to 5 in. diameter, and $\frac{1}{32}$ in. for larger pins.

42.—Pins and Rollers.—Pins and rollers shall be turned accurately to gauges, and shall be straight, smooth and entirely free from flaws.

43.—Pilot Nuts.—At least one pilot and driving nut shall be furnished for each size of pin for each structure.

44.—Screw Threads.—Screw threads shall make tight fits in the nuts, and shall be United States standard, except at the ends of pins and for bolts more than $1\frac{1}{4}$ in. in diameter, for which six threads per inch shall be used.

45.—Annealing.—Steel, except in minor details, which has been partially heated shall be properly annealed.

46.—Steel Castings.—All steel castings shall be annealed.

47.—Welds.—Welds in steel will not be allowed.

48.—Bed-Plates.—Expansion bed-plates shall be planed true and smooth. Cast wall-plates shall be planed at top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

49.—Shipping Details.—Pins, nuts, bolts, rivets and other small details shall be boxed or crated.

50.—Weight.—The weight of every piece and box shall be marked on it in plain figures.

Painting.

51.—Shop Painting.—Steelwork, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

52.—In riveted work, the surfaces coming in contact shall be painted before being riveted together.

53.—Pieces and parts which are not accessible for painting after erection, shall have two coats of paint before leaving the shop.

54.—Steelwork to be embedded in concrete shall not be painted.

55.—Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

56.—Machine-finished surfaces shall be coated with white lead and tallow before shipment, or before being put out into the open air.

57.—*Field Painting.*—After the structure is erected, the metalwork shall be painted thoroughly and evenly with an additional coat of paint, mixed with pure linseed oil, of such quality and color as may be selected.

Inspection and Testing.

58.—The manufacturer shall furnish all facilities for inspecting and testing the weight, quality of material and workmanship. He shall furnish a suitable testing machine for testing the specimens, as well as prepare the pieces for the machine, free of cost.

59.—When an inspector is furnished by the purchaser, he shall have full access at all times to all parts of the works where material under his inspection is manufactured.

60.—The purchaser shall be furnished with complete copies of mill orders, and no material shall be rolled and no work done before he has been notified as to where the orders have been placed, so that he may arrange for the inspection.

61.—The purchaser shall also be furnished with complete shop plans, and must be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect the material and workmanship.

62.—Complete copies of shipping invoices shall be furnished to the purchaser with each shipment.

63.—If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser.

Full-Sized Tests.

64.—Full-sized parts of the structure may be tested at the option of the purchaser. If tested to destruction, such material shall be paid for at cost, less its scrap value, if it proves satisfactory.

65.—If it does not stand the specified tests, it will be considered rejected material, and be solely at the cost of the contractor, unless he is not responsible for the design of the work.

66.—In eye-bar tests, the ultimate strength, the elastic limit and the elongation in 10 ft., unless a different length is called for, shall be recorded.

67.—In transverse tests, the lateral and vertical deflections shall be recorded.

APPENDIX A.

Extracts from the Following Building Laws.

City.	Year.
Buffalo	1896
St. Louis	1897
Philadelphia	1899
New York	1899
Chicago	1900
Boston	1900
Minneapolis	1900
Milwaukee	1901
Baltimore	1901
District of Columbia	1902

In Table 5 and all following tables:

S = Stress, in pounds per square inch;

L = Length, in inches (except in Table 15, where L = length in feet);

D = External diameter, or least side of rectangle, in inches;

R = Least radius of gyration of column section.

TABLE 5.—MINIMUM LIVE LOADS FOR FLOORS AND ROOFS.

Structure.	Pounds per Square Foot.									
	New York.	Chicago.	Philadelphia.	Boston.	Buffalo.	Minneapolis.	Milwaukee.	District of Columbia.	Baltimore.	St. Louis.
Dwellings—one or two families.	60	40	70	50	40	50	40	50	75	70
Lodging houses, apartment houses, tenement houses, hotels, etc.	60	40	70	50	70	50	40	50	...	70
Halls, dining-rooms, cafés, offices, etc., in hotels and apartment-houses	...	...	...	...	...	...	...	75	...	...
Office buildings, first floor	150	100	100	100	70	...	60	75	...	150
Office buildings, above first floor	75	100	100	100	70	...	60	75	...	70
Halls and lobbies in office buildings	...	...	...	...	...	...	...	110	...	...
Public assembly rooms: churches, theaters, etc.	90	100	120	150	100	100	80	110	150	120
Schools	75	100	120	150	100	100	50	75	150	120
Machine shops, armories, drill-rooms, etc.	...	...	...	250	...	250	250	...	...	...
Light manufacturing and retail stores and storehouses	120	100	120	...	120	100	100	110	150	150
Heavy storehouses, warehouses, livery stables, etc.	150	100	150	250	150	...	...	200	200	150
Stairways	...	...	...	...	...	...	100*	...	...	...
Sidewalks	300	...	...	...	...	350	...	...	...	...
Roofs, per square foot of superficial surface	50†	...	30	25§	...	50	30	25	30	50
Roofs, per square foot of horizontal projection	30‡	25	...	...	40	...	...	...	...	....

Wind Loads.

	New York.	Chicago.	Philadelphia.	Boston.	Buffalo.	Minneapolis.	Milwaukee.	District of Columbia.	Baltimore.	St. Louis.
Per square foot of elevation	30	30	30	30	30	...	30	30	...	30
			25 lb. at tenth story. 2½ lb. more for each story above. 2½ lb. less " " " below. 35 lb. maximum.				30 lb. at twelfth story. 2½ lb. less at each lower story.			

* Lower supports to carry two-thirds of total weight.
† Pitch less than 20 degrees.
‡ Pitch more than 20 degrees. § Also a horizontal wind pressure of 30 lb. per sq. ft.

TABLE 6.—Permissible Reduction of Live Loads.

Live Loads under Foundations in Buildings More than Three Stories High.

Structure.	New York.	Chicago.	District of Columbia.	St. Louis.
Warehouses and factories	100%	For many-storied buildings use actual average loads, and not theoretical or occasional loads.	100%	For office buildings: a live load of 10 lb. per sq. ft. on all floors. For mercantile buildings: a live load of 20 lb. per sq. ft. on all floors.
Stores and buildings for light manufacturing purposes	75 "		75 "	
Churches, schoolhouses and places of public assembly	75 "		90 "	
Office buildings, hotels, dwellings, apartment houses, tenement houses, stables, etc. of masonry construction	60 "		60 "	
do. do. of steel skeleton construction	60 "		75 "	

Live Loads on Columns.

New York.		St. Louis.	
In dwellings, office buildings, stores, stables and public buildings more than five stories in height.	For roof and top-floor columns full live load. For each succeeding lower floor columns a reduction of 5% until 50% of live load remains, which is minimum.	In office and mercantile buildings.	Attic columns: full live load. Basement columns: 80% of live load and proportionately for intermediate floors.

Live Loads on Girders (St. Louis only).

	80% of full live load. (Does not apply to beams.)

Philadelphia Law: For all tenant houses, hotels, apartment houses, hospitals and office buildings, the live loads on columns, girders and foundations, may be estimated by the formula, $X = 100 - \frac{1}{2}\sqrt{A}$, and for light manufacturing buildings by the formula, $X = 100 - \frac{1}{4}\sqrt{A}$, in which X = the percentage of live load to be used, and A = area carried by any girder, column or foundation.

For permissible loads in Milwaukee Law, see Table 7.

TABLE 7.—From Milwaukee Building Law.

Permissible Live Load, in Pounds per Square Foot of Floor Area, for Columns and Foundations, in Hotels, Apartment, Tenement and Lodging Houses and Office Buildings.

	Number of Stories in Building.									
	12	11	10	9	8	7	6	5	4	3
Roof	30									
12th Story	50	30								
11th "	35	50	30							
10th "	25	35	50	30						
9th "	20	25	35	50	30					
8th "	15	20	25	35	50	30				
7th "	10	15	20	25	35	50	30			
6th "	10	10	15	20	25	35	50	30		
5th "	5	10	10	15	20	25	35	50	30	
4th "	5	5	10	10	15	20	25	35	50	30
3d "	5	5	5	5	10	15	20	25	35	50
2d "	0	0	0	5	5	10	15	20	25	35
1st "	0	0	0	0	0	5	5	15	20	25
Totals	210	205	200	195	190	185	180	175	160	140

If the first or any other story is used for a store, hall, or for other business purposes, the full live load of 100 lb. per sq. ft. shall be considered as acting on the supporting columns.

TABLE 8.—BEARING CAPACITY OF DIFFERENT KINDS OF SOILS.

Bearing Material.	Tons per Square Foot.							
	New York.	Chicago.	Phila.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
Soft clay	1	......	......	......	......	1	1	According to test with a maximum for any soil of 3 tons per sq. ft.
Ordinary clay with wet sand	2	1½	......	......	......	......	2	
Dry clay	3	2¼	......	3½	3½	2	3	
Dry sand	3	2	......	......	3½	2	3	
Hard clay	4	......	3½	......	......	4	4	
Firm coarse sand	4	......	......	......	......	4	4	
Clay and gravel	......	1¾	......	......	......	......	......	
Sand and gravel	......	......	3½	......	......	......	......	
Cemented gravel	......	......	6	......	......	......	......	
Gravel and sand (well cemented)	......	......	......	......	......	3	......	
Rock, through earth (open caissons)	8	......	......	......	......	......	......	
Firm gravel or hard clay (through earth)	10	......	......	......	......	......	......	
Rock, through earth (pneumatic caissons)	15	......	......	......	......	......	......	
Concrete in foundations	......	4	......	4	4	......	......	
Dimension stones in foundations	......	5	......	6	6	......	......	
Dressed foundation stones, in cement mortar	......	7	......	7	7	......	......	
	Tons.							
Maximum pressure on one pile	20*	25	20	25	......	......*	25*	......
	Allowable fiber stress, in pounds per square inch.							
Steel beams in concrete, in foundations	......	16 000	......	16 000	......	......	......	......
Oak timber grillage on piles	......	1 200	......	1 200	......	......	......	......

* Safe sustaining power of one pile, in tons, is:

$$S = \frac{2 \times \text{weight of hammer, in tons} \times \text{height of fall, in feet}}{\text{Penetration, in inches (under last blow)} + 1 \text{ in.}}$$

TABLE 9.—Bearing Capacity of Materials.

Bearing Material.	Tons per Square Foot.								
	New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
Concrete: Portland cement 1, sand 2, stone 4	16½	4	......	......	......	......	.....	16½	.....
Concrete: Portland cement 1, sand 2, stone 5	15	......	15	......	......	......	.....	15	.....
Concrete: Rosendale cement 1, sand 2, stone 4	9	......	......	......	......	......	.....	9	.....
Concrete: Rosendale cement 1, sand 2, stone 5	8	......	......	......	......	......	.....	8	.....
Rubble in Portland cement mortar	10	......	10	......	......	......	.....	10	.....
Rubble in Rosendale cement mortar	8	......	......	......	5	5	.....	8	.....
Rubble in lime and cement mortar	7	......	8	......	......	......	.....	7	.....
Rubble in lime mortar	5	......	5	......	......	......	.....	5	.....
Brickwork in Portland cement mortar	18	12½	15	15	12	12	18	18	15
Brickwork in Rosendale cement mortar	15	9	......	......	5-9	5-9	.....	15	.....
Brickwork in lime and cement mortar	11½	......	12	12	......	......	14⅖	11½	11½
Brickwork in lime mortar	8	6½	8	8	3-6	4-6	10⅘	8	8
Granite	72-173	......	......	60	......	......	.....	72-173	.....
Marble and limestone	43-166	......	......	40	......	70	.....	43-166	.....
Sandstone	29-115	......	......	30	......	30	.....	29-115	.....
Bluestone	144	......	......	......	......	......	.....	144	.....
Slate	72	......	......	......	......	......	.....	72	.....
Hard-burned brick in piers, cement mortar	......	......	......	13	......	......	.....	......	.....
Hard-burned brick in piers, lime and cement mortar	......	......	......	10	......	......	.....	......	.....
Hard-burned brick in piers, lime mortar	......	......	......	7	......	......	.....	......	.....

TABLE 10.—PERMISSIBLE UNIT STRESSES IN MATERIALS.

Material.	POUNDS PER SQUARE INCH.								
	New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
Tension.—Rolled steel	16 000	15 000	......	15 000	16 000	16 000	16 000	16 000	......
Cast "	16 000	......	......	......	......	......	......	16 000	......
Medium "	......	......	16 250	......	......	......	......	......	......
Soft "	......	......	14 500	......	......	......	......	......	......
Cast iron	3 000	......	......	......	3 000	......	3 000	3 000	......
Wrought "	......	......	12 500	......	12 000	12 000	......	12 000	......
Yellow pine	1 200	......	1 800	......	......	......	......	1 200	......
White "	800	......	......	......	......	......	......	800	......
Spruce	800	......	1 400	......	......	......	......	800	......
Oak	1 000	......	......	......	......	......	......	1 000	......
Hemlock	600	......	1 000	......	......	......	......	......	......
Compression.—Rolled steel	16 000	15 000	......	12 000	......	......	......	16 000	12 000
Cast steel	16 000	......	......	......	......	......	......	16 000	......
Cast iron	16 000	10 000	11 500	......	13 000	......	......	16 000	......
Steel pins and rivets (bearing)	20 000	20 000	22 000	18 000	15 000	15 000	......	20 000	......
Wrought iron	......	......	8 300	......	......	......	......	12 000	......
Soft steel	......	......	14 500	......	......	......	......	......	......
Medium "	......	......	16 250	......	......	......	......	......	......
Oak (with grain)	900	......	......	......	......	......	......	900	......
" (across ")	800	250	......	250	......	......	......	800	......
Yellow pine (with grain)	1 000	......	750	......	......	......	......	1 000	......
Yellow pine (across grain)	600	250	550	250	......	......	......	600	......
White pine (with grain)	800	......	......	......	......	......	......	800	......
White pine (across grain)	400	150	......	150	......	......	......	400	......
Spruce (with grain)	800	......	500	......	......	......	......	800	......
" (across ")	400	150	300	150	......	......	......	400	......
Locust (with grain)	1 200	......	......	......	......	......	......	1 200	......
" (across ")	1 000	......	......	......	......	......	......	1 000	......
Hemlock (with grain)	500	......	350	......	......	......	......	......	......
" (across ")	500	......	250	......	......	......	......	......	......
Chestnut (with grain)	500	......	......	......	......	......	......	500	......
Chestnut (across grain)	1 000	......	......	......	......	......	......	1 000	......

TABLE 11.—PERMISSIBLE UNIT STRESSES IN MATERIALS.

Material.	Pounds per Square Inch.								
	New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
Shear—Steel	9 000	10 000	......	10 000	7 000	7 000	......	9 000	12 000
" Shop rivets and pins	10 000	......	11 000	......	9 000	9 000	9 000	10 000	*9 000
" Field rivets	8 000	......	8 800	......	8 000	8 000	7 500	8 000	7 000
" Field bolts	7 000	......	......	......	......	......	......	7 000	7 000
Soft steel	......	......	8 750	......	......	......	......	......	......
Med. "	......	......	10 000	......	......	......	......	......	......
Wrought iron	......	......	7 500	......	......	......	......	......	......
Cast "	8 000	......	......	......	......	......	......	8 000	......
Yellow pine (with grain)	70	100	100	100	......	......	......	70	......
" " (across ")	500	......	1 120	......	......	......	......	500	......
White " (with ")	40	80	......	80	......	......	......	40	......
" " (across ")	250	......	......	......	......	......	......	250	......
Spruce (with ")	50	80	70	80	......	......	......	50	......
" (across ")	320	......	750	......	......	......	......	320	......
Oak (with ")	100	150	......	150	......	......	......	100	......
" (across ")	600	......	......	......	......	......	......	600	......
Locust (with ")	100	......	......	......	......	......	......	100	......
" (across ")	720	......	......	......	......	......	......	720	......
Hemlock (with ")	40	......	60	......	......	......	......	......	......
" (across ")	275	......	620	......	......	......	......	......	......
Chestnut (" ")	150	......	......	......	......	......	......	......	......

* For Rivets—Pins 12 000 lb. per sq. in.

Table 12.—Permissible Unit Stresses in Materials.

Material.	New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
	Pounds per Square Inch.								
Bending—Rolled steel beams...	16 000	16 000	......	16 000	16 000	16 000	16 000	16 000	16 000
Rolled steel pins, rivets and bolts....	20 000	22 500	......	22 500	......	......	......	20 000	20 000
Riveted steel girders, net section.........	14 000	15 000	......	......	13 500	13 500	12 500	14 000	12 000*
Cast iron (tension)...	3 000	2 500	3 750	2 500	......	......	3 000	3 000	......
Cast iron (compression)...............	16 000	......	......	8 000	......	......	13 500	16 000	......
Yellow pine..........	1 200	1 250	1 600	1 250	1 800	......	1 440	1 200	......
White pine..........	800	750	......	750	1 080	1 080	900	800	......
Spruce...............	800	750	1 100	750	......	......	......	800	......
Oak.................	1 000	1 000	......	1 000	1 350	1 350	1 080	1 000	......
Locust..............	1 200	......	......	......	......	......	......	1 200	......
Hemlock.............	600	......	900	......	1 080	1 080	......	......	......
Chestnut............	800	......	......	......	......	......	......	800	......
Washington fir.......	......	......	......	......	......	1 800	......	......	......
Granite..............	180	......	......	......	......	......	......	......	......
Gneiss...............	150	......	......	......	......	......	......	......	......
Limestone	150	......	......	......	......	......	......	......	......
Slate	400	......	......	......	......	......	......	......	......
Marble	120	......	......	......	......	......	......	......	......
Sandstone...........	100	......	......	......	......	......	......	......	......
Bluestone...........	300	......	......	......	......	......	......	......	......
Portland cement 1:2:4	30	......	......	......	......	......	......	......	......
" " 1:2:5	20	......	......	......	......	......	......	......	......
Natural " 1:2:4	16	......	......	......	......	......	......	......	......
" " 1:2:5	10	......	......	......	......	......	......	......	......
Brick (common).....	50	......	......	......	......	......	......	......	......
Brickwork in cement	30	......	......	......	......	......	......	......	......

*Compression Flange (gross section.)

TABLE 13.—PERMISSIBLE UNIT STRESSES IN MATERIALS.

Material.		Pounds per Square Inch.								
		New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
Columns:										
Yellow pine	$\frac{L}{D}$ = 10..	820				1 000	1 000	Rectangular Columns $S = 900 \div \left(1 + \frac{L^2}{250\,D^2}\right)$	820	....
	12..	784	1 000		1 000				784	....
	15..	730							730	....
	20..	640		$S = 750 - 7.5\,\frac{L}{D}$		$S = 1\,000 - 10\,\frac{L}{D}$	$S = 1\,000 - 10\,\frac{L}{D}$	Round " $S = 900 \div \left(1 + \frac{L^2}{185\,D^2}\right)$	640	....
	25..	550	875		875				550	....
	30..	460							460	....
	40..	...	750		750				...	....
	45..	...	625		625				...	....
	50..	...	500		500				...	....
White pine and spruce	$\frac{L}{D}$ = 10..	650	625		625	700	700	Rectangular Columns $S = 600 \div \left(1 + \frac{L^2}{250\,D^2}\right)$	650	....
	12..	620							620	....
	15..	575	500	$S = 500 - 5\,\frac{L}{D}$	500				575	....
	20..	500				$S = 625 - 6\,\frac{L}{D}$	$S = 625 - 6\,\frac{L}{D}$	Round " $S = 600 \div \left(1 + \frac{L^2}{185\,D^2}\right)$	500	....
	25..	425							425	....
	30..	350							350	....
	35..	...	375		375				...	....
	45..	...	250		250				...	....
	50..	...							...	....
Oak	$\frac{L}{D}$ = 10..	730				800	800	Rectangular Columns $S = 800 \div \left(1 + \frac{L^2}{250\,D^2}\right)$	730	....
	12..	696	750		750				696	....
	15..	645							645	....
	20..	560		$\left(S = 350 - 3.5\,\frac{L}{D} \text{ for hemlock}\right)$				Round " $S = 800 \div \left(1 + \frac{L^2}{185\,D^2}\right)$	560	....
	25..	475	650		650	$S = 750 - 7.5\,\frac{L}{D}$	$S = 750 - 7.5\,\frac{L}{D}$		475	....
	30..	390							390	....
	40..	...	560		560				...	....
	45..	...	470		470				...	....
	50..	...	375		375				...	....

TABLE 14.—Permissible Unit Stresses in Materials.

Material.	Pounds per Square Inch. New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.
Columns:								
Cast iron....	$S = 11\,800 - 80\frac{L}{R}$	10 000 (reduced by Gordon's formula.) $\left(\frac{L}{D} = 20\right.$ is maximum.$\left.\right)$	$S = \frac{11\,700}{1 + \frac{L^2}{400\,D^2}}$	See Table 15.	Rectangular columns $S = 14\,000 \div \left(1 + \frac{L^2}{850\,D^2}\right)$ Round columns $S = 14\,000 \div \left(1 + \frac{L^2}{600\,D^2}\right)$	Rectangular columns $S = 14\,000 \div \left(1 + \frac{L^2}{850\,D^2}\right)$ Round columns $S = 14\,000 \div \left(1 + \frac{L^2}{600\,D^2}\right)$	Rectangular columns $S = 8\,000 \div \left(1 + \frac{L^2}{400\,D^2}\right)$ Round columns $S = 8\,000 \div \left(1 + \frac{L^2}{600\,D^2}\right)$	$S = 11\,800 - 80\frac{L}{R}$
Steel.........	$S = 15\,200 - 58\frac{L}{R}$*	15 000 reduced by approved modern formula.	Soft steel $S = 14\,500 \div \left(1 + \frac{L^2}{13\,500\,R^2}\right)$ Medium steel $S = 16\,250 \div \left(1 + \frac{L^2}{11\,000\,R^2}\right)$	12 000 reduced by approved modern formula.	$\frac{L}{R} > 90$ $S = 17\,100 - 57\frac{L}{R}$ $\frac{L}{R} < 90$ $S = 12\,000$	$\frac{L}{R} > 90$ $S = 17\,100 - 57\frac{L}{R}$ $\frac{L}{R} < 90$ $S = 12\,000$	$S = \frac{15\,000}{1 + \frac{L^2}{36\,000\,R^2}}$	$S = 15\,200 - 58\frac{L}{R}$
Columns: Maximum limit $\frac{L}{R}$..	120 (steel and iron)	....	140 or	....	....	....	....	120 (steel and iron)
Maximum limit $\frac{L}{D}$..	30 (timber)	20 (cast iron)	45	....	40 (steel)	40 (steel)	....	30 (timber)
Modulus of elasticity:								
Steel	......	....	....	29 000 000	....	....	....	......
Iron..........	......	....	....	27 000 000	....	....	....	......
White pine....	......	....	....	750 000	....	....	....	......
Spruce	......	....	....	900 000	....	....	....	......
Yellow pine....	......	....	....	1 300 000	....	....	....	......
White oak.....	......	....	....	860 000	....	....	....	......

* For compression members in pin-connected trusses, use 75% of working stresses for columns.

Table 15.—From Boston Building Law.

Working Stresses for Columns of Cast Iron.						
	Round Columns.			Rectangular Columns.		
$\frac{L}{D}$	S			S		
	Square-faced bearings.	Round and faced bearings.	Round bearings.	Square-faced bearings.	Round and faced bearings.	Round bearings.
1.0	8 480	7 870	7 350	8 810	8 320	7 870
1.5	7 120	6 220	5 530	7 670	6 870	6 220
2.0	5 810	4 810	4 100	6 490	5 530	4 810
2.5	4 710	3 720	3 080	5 420	4 410	3 720
3.0	3 820	2 920	2 360	4 520	3 540	2 920

Note.—In this table L = Length, in feet.

Table 16.—Weights of Building Materials, etc., in Pounds per Cubic Foot.

Material.	Weight.
Paving brick	150
Common building brick	120
Soft building brick	100
Granite	170
Marble	170
Limestone	160
Sandstone	145
Slag	40
Gravel	120
Slate	175
Sand, clay and earth (dry)	100
Sand, clay and earth (wet)	120
Mortar	100
Stone concrete	130–150
Cinder concrete	70
Paving asphaltum	100
Plaster of paris	140
Glass	160
Snow, freshly fallen	10
Snow, wet	50
Spruce	25
Hemlock	25
White pine	25
Douglas fir	30
Yellow pine	40
White oak	50
Common brickwork	100–120
Rubble masonry	130–150
Ashlar masonry	140–160
Cast iron	450
Wrought iron	480
Steel	490
Plaster, ceiling	10 to 15 lb. per sq. ft.

TABLE 17.—Typical Electric Traveling Cranes.

Capacity in tons.	Span.	Wheel base.	Maximum wheel load, in pounds.	s.	v.	Weight of Rail for:	
						Plate Girders.	Beams.
5	40	8 ft. 6 in.	12 000	10 in.	7 ft.	40 lb. per yd.	40
	60	9 " 0 "	13 000	"	"	40 "	40
10	40	9 " 0 "	19 000	"	"	45 "	40
	60	9 " 6 "	21 000	"	"	45 "	40
15	40	9 " 6 "	26 000	"	"	50 "	50
	60	10 " 0 "	29 000	"	"	50 "	50
20	40	10 " 0 "	33 000	12 in.	8 ft.	55 "	50
	60	10 " 6 "	36 000	"	"	55 "	50
25	40	10 " 0 "	40 000	"	"	60 "	50
	60	10 " 6 "	44 000	"	"	60 "	50
30	40	10 " 6 "	48 000	"	"	70 "	60
	60	11 " 0 "	52 000	"	"	70 "	60
40	40	11 " 0 "	64 000	14 in.	9 ft.	80 "	60
	60	12 " 0 "	70 000	"	"	80 "	60
50	40	11 " 0 "	72 000	"	"	100 "	60
	60	12 " 0 "	80 000	"	"	100 "	60

1.—Wheel-load can be assumed as distributed in top flange, over a distance equal to depth of girder, with a maximum limit of 30 in.

2.—In addition to the vertical load, the top flanges of the girder shall withstand a lateral loading of two-tenths of the lifting capacity of the crane, equally divided between the four wheels of the crane.

s = Side clearance from center of rail.

v = vertical " " top " "

3.—The top flanges of the crane girders shall not be of a smaller width than one-twentieth of their unsupported length.

TABLE 18.—Thickness of Spruce and White Pine Plank for Floors.

Span, in feet.	Thickness, in Inches, for Various Loads per Square Foot of Plank.																
	lb. 30	lb. 40	lb. 50	lb. 75	lb. 100	lb. 125	lb. 150	lb. 175	lb. 200	lb. 225	lb. 250	lb. 275	lb. 300	lb. 325	lb. 350	lb. 375	lb. 400
4	0.9	1.1	1.2	1.5	1.7	1.9	2.1	2.2	2.4	2.5	2.7	2.8	2.9	3.1	3.2	3.3	3.4
5	1.2	1.4	1.5	1.9	2.1	2.4	2.6	2.8	3.0	3.2	3.4	3.5	3.7	3.8	4.0	4.1	4.3
6	1.4	1.6	1.8	2.2	2.6	2.9	3.1	3.4	3.6	3.8	4.0	4.2	4.4	4.6	4.8	4.9	5.1
7	1.7	1.9	2.1	2.6	3.0	3.3	3.7	3.9	4.2	4.5	4.7	4.9	5.2	5.4	5.6	5.8	6.0
8	1.9	2.2	2.4	3.0	3.4	3.8	4.4	4.5	4.8	5.1	5.4	5.7	5.9	6.1	....	....	....
9	2.1	2.5	2.7	3.4	3.9	4.3	4.7	5.1	5.4	5.8	6.1	....	....	....	....	....	....
10	2.4	2.7	3.1	3.7	4.3	4.8	5.2	5.6	6.0	....	....	....	....	....	....	....	....
11	2.6	3.0	3.4	4.1	4.7	5.3	5.8	....	....	....	....	....	....	....	....	....	....
12	2.9	3.3	3.7	4.5	5.2	....	....	....	....	....	....	....	....	....	....	....	....
13	3.1	3.6	4.0	4.9	5.6	....	....	....	....	....	....	....	....	....	....	....	....
14	3.4	3.9	4.3	5.3	6.1	....	....	....	....	....	....	....	....	....	....	....	....

For yellow pine use nine-tenths of the above thicknesses.

Vol. XXX. SEPTEMBER, 1904. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

LAKE CHEESMAN DAM AND RESERVOIR.

Discussion.*

BY G. S. WILLIAMS, M. AM. SOC. C. E.

Mr. Williams. G. S. WILLIAMS, M. AM. SOC. C. E.—To the speaker, outside of the very interesting information in regard to the methods of construction and the details of the design of the Lake Cheesman Dam, the most valuable part of the paper is that by Mr. Woodard. Engineers have been waiting a good while for an American to attack this problem of the analysis of the relative strains set up in a dam which combines the arch and the gravity sections, and they should feel grateful to the authors for having brought out a solution of the problem. It may be remarked, however, that where a dam abuts against vertical walls the action becomes more nearly akin to that of a plate supported on three edges than to that of a beam fixed at one end.

In looking over the paper and the illustrations, other possible solutions of this problem have been suggested, and it may be well to mention two or three treatments which might have been introduced, perhaps with no better results, but at least they should be interesting.

Undoubtedly, all have been impressed with the fact that, to start with, there were two solid abutments, and, having that for a dam of moderate span, and by moderate is meant less than 400 or 500 ft., there seems to be no other type that combines greater stability with a mini-

*Continued from August, 1904, *Proceedings*. See March, 1904, *Proceedings*, for paper on this subject by Charles L. Harrison, M. Am. Soc. C. E., and Silas H. Woodard, Assoc. M. Am. Soc. C. E.

mum of material than an arch, pure and simple, with the gravity section eliminated. Mr. Williams.

An arch, to act as an arch, must be designed so as to have a considerable degree of elasticity, or the arch action will fail; that is to say, it is not possible to get arch action with very thick material unless the stresses are extremely high, and, therefore, the authors were fully justified in eliminating from consideration as an arch the lower section of the dam.

In casting up the possibilities at the authors' site, without going into the matter very definitely, it appears that the controlling point for an arch dam was at an elevation of about 150 ft., where, on account of the span, a radius of about 200 ft. would be necessary. Limiting the stresses to those computed by the authors for the existing structure, it appears that an arch of about 25 ft. thickness would be required, and, starting with that, it would decrease to 1 ft. or any greater thickness that was desired, at the top, without increasing the stresses beyond those which the authors allowed in the structure as they designed it. Their stress, as they show it, is about 240 lb. per sq. in., or 35 000 lb. per sq. ft., and, taking that as a limiting stress, which, all will agree, is a perfectly safe stress for masonry of this character, a dam might have been designed for this place having a maximum thickness of 25 ft., and from that reducing to any desired thickness at the top; that design being a simple vertical cylindrical arch above the 150-ft. contour, and being of smaller radius below.

Another solution that appears in this case, is to have designed the dam as an inverted cone, and the spot would have been quite favorable for such a treatment. It will be recalled that the thrust of an arch under normal loads is equal to the pressure on the extrados into the radius of the extrados—not that of the center line, as is often incorrectly assumed—so that, by varying the radius, the thickness, or the total thrust to be taken by the arch, may be varied. Starting in that way, it would be possible to make the dam of equal strength, but much less than 25 ft. in thickness at the base, if such were desired. Whether or not it would be safe involves consideration of the permeability of the masonry, and that is possibly an open question with some.

Still another treatment would have been to make the base of the dam a segment of a sphere, and, recalling that the thrust in a spherical dome is only one-half of that in a cylinder of equal radius, still less material might thus be used in the base of the dam, and then, as the upward thrust of the sphere would have to be absorbed by the weight of the material above it, the radius of the sphere would be limited to that giving a thrust not greater than the weight of the material above its equator.

These are only offered as possible solutions, and there may have been reasons, other than structural ones, for not adopting such departures from former general practice.

Mr. Williams. Not long ago it fell to the speaker's lot to design a dam for a spot which seemed to be equally well, if not even better, suited for the arch solution of the problem, and, as illustrating a purely arch design, that location, the structure designed, and the structure built, are shown in Plates XXXIX, XL and XLI. This site was in the vicinity of Ithaca, N. Y., about 2 miles from the center of the town, and the work was designed for the Ithaca Water-Works Company.

The Six-Mile Creek Dam.

Location and Conditions.—Six-Mile Creek, a stream having a quite precipitous drainage area of about 48 sq. miles above the point in question, there passes in a northerly direction through a gorge or miniature cañon about 500 ft. long and 90 ft. wide. The location selected for the dam, Fig. 1, Plate XXXIX, was near the upper end of this gorge, where the rock on the east side rises to a height of 90 ft. above the bed of the stream, overhanging in its rise 4 or 5 ft., and on the west side a similar wall, receding 6 ft. in its height, rises 70 ft. above the bed. On both sides, the rock was surmounted by a heavy deposit of drift clay, containing boulders, but quite impervious, and rising with a slope of nearly 30° for 50 or 100 ft. more. The rock was the bluish-gray shale so common in that region, traversed at intervals of from a few inches to several feet with nearly parallel fissures, the sides of which, except near the exposed walls of the gorge, were in close contact, and, where open, the seams were filled with fine clay washed in from the covering beds. The planes of the fissures were also nearly parallel to the axis of the gorge. On the exposed faces the rock was weathered for a depth of about 6 in. to a varying extent, thereby showing very clearly its stratified character; but, where the weathered surface was removed, the faces of the fissures showed a smooth, dense rock without apparent horizontal seams, except at intervals, usually of several feet. The bottom of the gorge was covered to a depth of about 6 ft. by a deposit of sand and gravel, caused by the construction, a few years previously, of a small dam at its outlet. The bed itself was of shale rock, similarly fissured and nearly level throughout three-fourths of the width of the gorge, and rising in steps of about 4 ft. near the west wall.

As the location was only a short distance above the city, and, as a failure of the structure would involve considerable financial loss, not only to the citizens generally, but especially to the Water Company, whose pumping station was on the bank of the stream less than a mile below, it was at once apparent that a type of dam should be selected which would be stable against all possible contingencies.

The conditions being such as to call, first of all, for an overfall dam, and the seams in the bottom running longitudinally of the gorge and thus possibly permitting percolation and an upward press-

FIG. 1.—SITE OF SIX-MILE CREEK DAM.

FIG. 2.—BUILDING THE SIX-MILE CREEK DAM.

Mr. Williams.

ure on the base, were to the speaker the strongest of arguments against the adoption of a gravity section of the ordinary type in this location.

Were the problem presented of carrying a roadway, even for the heaviest kind of traffic, across the gorge in question, no one would for a moment think of laying pipes or building culverts along the bottom and filling the chasm level full of masonry on top of them, nor would he put in a series of piers and connect them with short plate girders; but the one obvious and correct solution would be to span the depression with an arch either of metal, wood or masonry. Bearing in mind that the arch, under vertical moving loads, can never be in equilibrium, but must always resist varying bending moments, and that under normally applied uniform forces a circular rib will be in equilibrium, and subject to no bending moments, except those possibly induced by temperature changes and the compression of the material itself, which are also similarly possible in arch bridges, the propriety of applying the concrete arch to the problem becomes at once apparent, for it will be seen that the only possible means of failure for a circular arch under normal uniform forces is by the ultimate crushing of the material; and the conditions of the permeability of the base or foundation rock and consequent upward pressure underneath the dam, or a side pressure at the ends, have no influence upon the stability of such a structure in this location. The only possible ways for it to be destroyed by natural means are by the yielding of the abutments to such an extent as to cause the ultimate crushing strength of the material to be exceeded, or by the direct application of such a pressure as to bring about such a stress.

Design.—One of the chief criticisms directed against arch dams has been that, by reason of the rigidity of the base, the arch action could not be developed in their lower part, and, while the speaker is not one of those who would argue that a barrel is weaker against external pressure by reason of having the heads in it, yet, to overcome this objection, and avoid as far as possible stresses of opposite signs acting at right angles to each other, a condition which certainly weakens the material's ultimate capacity to resist either one, recourse was had to a design similar to that introduced in an egg-ended boiler, and the base, as shown by Fig. 30, was made of the form of a portion of a torus or ring. The whole structure was to be 90 ft. in height, and the radius of the vertical curve of the base was 20 ft., selected so that the upward thrust at Elevation 185 would never exceed the downward pressure transmitted through the material above. By this construction it became possible to utilize the bed of the stream as an abutment at $t\ s$, and still permit of elastic deformations and true arch or dome action near or at the base. By inclining the radius at the junction of the torus with the superimposed cylinder at $b\ r$, an up-stream

Mr. Williams. thrust at this point was obtained from the former tending to oppose the pressure of the water and decrease the horizontal circumferential thrust in the cylinder. Similarly, the inclination of that portion of the structure above *b r* also introduces a thrust up stream, acting

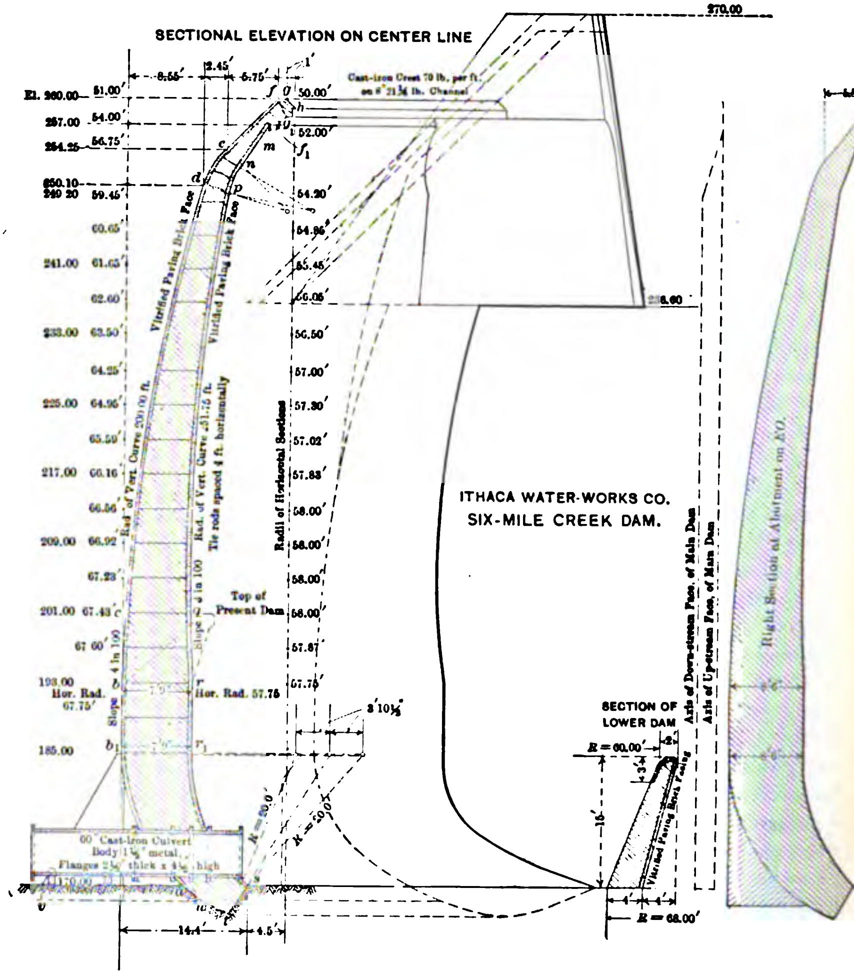

FIG. 80.

likewise to decrease the horizontal circumferential thrust. Above this plane, to Elevation 250.10, the section is made up of a series of frustums of conical shells. From 250.10 to 254.25 it is a segment of a ring, and from 254.25 to the crest at 260.0 it is a segment of a conical dome. The radii of the extrados or up-stream face are shown on the left of the section, and of the intrados on the right. The maximum radius of the extrados was 67.75 ft. and that of the crest 50 ft. Mr. Williams.

The axes of the two faces are not coincident, that of the down-stream face being 2.25 ft. up stream from that of the up-stream face, thus making the dam somewhat thicker at the abutments than at the center.

The shape of the crest was selected for the following reasons:

First, a form was desired which would discharge a maximum quantity of water at heads above 2 ft., and the one selected has been found by experiment to approximate closely to such a condition.

Second, a form was desired which would readily permit of ice climbing it, and the slope of 45° adopted answers this requirement well.

Third, a form was desired which would insure positive, certain and continuous aeration of the region behind the sheet, and the prevention of the formation of even a minute vacuum there, and the large space between the face of the dam and the falling water, in free communication with the air outside, effectually precludes the occurrence of a condition which, the speaker believes, has been, to no small extent, responsible for the failure of overfall dams in the past.

Fourth, a form was desired which would deliver the overfalling sheet well away from the toe of the section, and an inspection of Fig. 30 shows that this condition has been met.

As a further protection to the bottom, and also to insure a uniform upward thrust at $b\ r$, whether the pond were full or in flood, a second dam, 15 ft. high, was to be constructed about the middle of the gorge, 170 ft. down stream from the main dam, the overfall from which would be received in a pool formed by the old low dam already mentioned, which is 210 ft. farther down stream. This lower or middle dam was to be a segment of a frustum of a cone with a crest radius of 60 ft.

Computation of Stresses.—For preliminary purposes, the well-known formula, $T = p\ R$, wherein $T =$ the thrust or pull in the sheet, $p =$ the normal force, and $R =$ the radius of the face to which the force is applied, may be used, and, were the section cylindrical, p would be the water pressure and R the horizontal radius, and this formula would be rigidly applicable for the determination of the arch stresses. As, in the present design, the faces are generally inclined, this fact must be taken into account, and the formula becomes $T' = p\ R$ sec. i, R being still the horizontal radius and i the angle of inclination of the face from the vertical. If the thickness be represented by F, then the unit thrust, $t = \frac{p\ R \text{ sec. } i}{F}$ for a section one unit high, omitting the effect of the inclination in producing a radial thrust opposite to T. As this counter thrust actually reduces T, it is evident that stresses

Mr. Williams. computed by the foregoing formula will be greater than those really existing in the horizontal circumferential direction. For a flood 10 ft. in depth above the crest of the dam, which requires a run-off of 353 cu. ft. per sq. mile per sec., while the largest flood on record in this stream gave less than 100 cu. ft. and would require about 4 ft. head, the approximate thrusts in the horizontal arches by the above formula are as given in Table 9.

TABLE 9.—APPROXIMATE STRESSES (IN EXCESS OF REAL STRESSES, EXCEPT ON OVERHANG NEAR CREST.)

Elevation.	Pressure of water, in pounds per square inch. p.	Horizontal radius, in inches. R.	$p\ R$ sec. i = thrust, in pounds. T.	Thickness, in inches. F.	Unit thrust, in pounds per square inch. t.
250	8.65	711	6 870	36	177
241	12.60	740	9 700	47.4	208
233	16.00	762	12 650	57	222
225	19.50	779	15 795	64.7	244
217	23.00	794	18 950	73	260
209	26.45	808	22 100	80	276
201	29.90	809	24 200	86	281
193	32.40	818	26 350	93	284

The thrusts in the torus base, being largely absorbed by the vertical arch of 240 in. radius, give much lower unit stresses.

For a final and more accurate determination of the stresses, the method used was as follows:

A vertical slice of the dam at the center, 1 in. thick at the up-stream face, was cut out by vertical radial planes and divided by planes normal to the up-stream face into 31 blocks, of which Nos. 1 to 7, inclusive, are on the overhang, 8 to 22 on the curved upper body, 23 is the cylinder, and 24 to 31 are on the torus base.

Beginning at the top, the force due to water pressure and that due to the weight of the block above the plane of its base are combined by a simple triangle of forces, Fig. 31, and the resultant, P, resolved into a horizontal component, P_h, and one normal to the base, P_n. For the next section, this resultant, P, is combined with the weight of the added block and the force due to pressure upon it, and a new resultant obtained which is resolved as above. Then, by

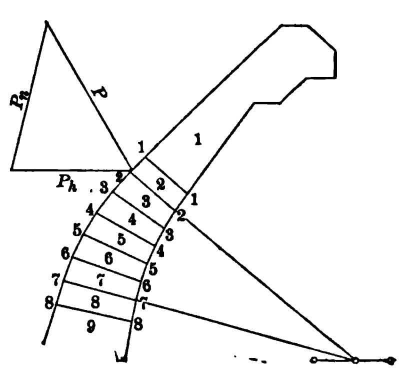

FIG. 31.

Mr. Williams.

Fig. 32, the forces acting on the block in question are:

P_w = the water pressure on the face of the block acting normally thereto;

P_n = the component of the total pressure, P, normal to the base;

P_{n-1} = the component of the total pressure, P, normal to the plane of the top of the block, which = P_n for the section next above;

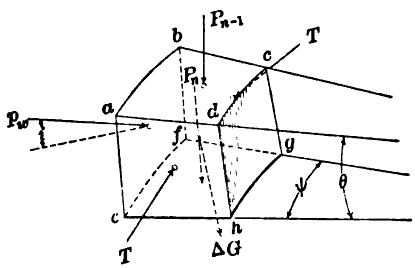

FIG. 32.

$\Delta\ G$ = the weight of the block;

T = the horizontal thrust in the arch ring;

R = the horizontal radius of the up-stream face;

θ = the angle which the top and bottom faces make with each other;

ψ = the angle which the side faces make with each other;

i = the angle which the normal to the up-stream face makes with the horizon.

Strictly, the slice should have been cut out between meridional planes, in which case its thickness, if 1 in. at the crest, would have been $\frac{67.75}{50.00}$ = 1.355 in. at the cylinder; or, being 1 in. thick at the cylinder, it would have been 0.738 in. at the crest. The effect of this correction would be to reduce slightly the components of G, but this is compensated for by taking a low value for the weight of the material, 140 lb. per cu. ft., and by neglecting to consider the weight of the metal in the structure.* At the base of Section 7 the theoretical thickness for a slice 1 in. thick at the cylinder would be 0.874 in., and the thinness of the sections in a radial direction at the top makes the error possibly introduced of small practical moment.

For equilibrium, by Fig. 32:

$$p_w - \left\{ (P_n + P_{n-1}) \sin. \frac{\theta}{2} - \Delta\ G \sin. i + 2\ T \sin. \frac{\psi}{2} \cos. i \right\} = 0,$$

or,

$$2\ T \sin. \frac{\psi}{2} \cos.\ i = p_w + \Delta\ G \sin.\ i - (P_n + P_{n-1}) \sin. \frac{\theta}{2}$$

If H = the total horizontal force carried by the horizontal arch;

F_v = the area of the vertical faces;

F_h = the area of the normal faces;

* The weight of the concrete alone, without the added boulders, was 141.4 lb. per cu. ft. The brick facing weighed 144.4 lb. per cu. ft. and the iron and steel averaged more than 0.8 lb. per cu. ft. of the entire mass, one-half of this weight being within 2 ft. of the crest.

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TABLE 10.—ANALYSIS OF STRESSES IN SIX-MILE CREEK DAM, (Slide-Rule

SECTION.		ELEVATION: FEET ABOVE CAYUGA LAKE = CITY DATUM.		TOTAL PRESSURE ABOVE BASE OF SECTION. P. POUNDS.		TOTAL PRESSURE NORMAL TO BASE OF SECTION. P_n. POUNDS.		WATER PRESSURE NORMAL TO FACE OF SECTION. P_w. POUNDS.		Weight of section. ΔG. Pounds.
No.	Location.	Base.	Center.	Full pond.	10-ft. flood.	Full pond.	10-ft. flood.	Full pond.	10-ft. flood.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	On overhang.	......	256.7	234	588	292	686	94.8	467	157.0
2		254.00	254.0	275	696	340	775	118.6	535	178.0
3		253.25	253.5	327	802	378	800	30.2	81	22.5
4		252.75	253.0	374	874	403	820	35.4	87.4	24.0
5		252.05	252.5	432	978	440	880	38.5	89	25.4
6		251.15	251.6	486	1 084	471	940	44.2	96	26.6
7		250.25	550.7	547	1 180	503	985	47.8	99	29.0
8		249.3	249.8	608	1 216	545	965	51.0	102	30.0
9	On curved upper body.	245.0	247.1	974	1 800	808	1 800	296.0	525	162.0
10		241.0	243.0	1 396	2 400	1 075	1 603	364.0	578	172.5
11		237.0	239.0	1 920	3 120	1 382	1 975	449.0	663	191.5
12		233.0	235.0	2 520	3 920	1 690	2 390	533.0	746	208.0
13		229.0	231.0	3 200	4 810	2 080	2 690	616.0	827	222.0
14		225.0	227.0	3 963	5 770	2 383	3 070	698.0	910	236.0
15		221.0	223.0	4 813	6 830	2 765	3 500	777.0	989	248.0
16		217.0	219.0	5 725	7 932	3 135	3 900	859.0	1 065	261.5
17		213 0	215.0	6 710	9 110	3 485	4 300	940.0	1 150	270.5
18		209.0	211.0	7 770	10 400	3 875	4 710	1 020.0	1 229	281.0
19		205.0	207.0	8 925	11 780	4 240	5 050	1 102.0	1 310	296.0
20		201.0	203.0	10 130	13 150	4 620	5 490	1 187.0	1 398	313.5
21		197.0	199.0	11 440	14 690	5 000	5 830	1 269.0	1 478	324.0
22		193.0	195.0	12 855	16 260	5 400	6 220	1 352.0	1 560	326.0
23	Cylinder.	185.0	189.0	15 825	19 490	6 050	6 900	2 950.0	3 202†	692.0
24	On torus base.	182.8	183.9	16 620	20 480	6 200	7 050	833.0*	884†	158.0
25		180.7	181.8	17 400	21 300	6 300	7 160	844.0*	894†	158.0
26		178.6	179.7	18 140	22 120	6 390	7 230	856.0*	907†	155.7
27		176.6	177.4	18 880	22 890	6 350	7 260	866.0*	918†	153.6
28		174.8	175.7	19 530	23 540	6 330	7 230	875.0*	925†	150.0
29		173.1	174.0	20 120	24 200	6 200	7 160	885.0*	933†	142.5
30		171.5	172.3	20 600	24 800	5 950	7 020	888.0*	938†	134.1
31		170.0	170.7	21 050	23 350	5 680	6 830	893.0*	942†	124.6
										Σ 5 863.5

* Component of water pressure on down-stream face subtracted.

§ Strictly, R, for center of gravity, should be used, since forces are combined there. By so doing the horizontal thrusts would be reduced about 2% at the top and 7½% at the cylinder.

Mr. Williams.

OMITTING INFLUENCE OF ATTACHMENTS AT SIDES AND BOTTOM.
Computations.)

Sine $\frac{\theta}{2}$	Sec. i.	Tan. i.	Radius of extrados of horizontal arch.§ R. Feet.	Areas. Vertical face of section. F_v. Square feet.	Areas. Base of section. F_b. Square inches.	Total Force Carried by Horizontal Arch. H. Pounds. Full pond.	Total Force Carried by Horizontal Arch. H. Pounds. 10-ft. flood.	Thrusts in Arches, in Pounds per Square Inch. Horizontal. $t = \frac{H R}{12 F_v}$ ‡ Full pond.	Thrusts in Arches, in Pounds per Square Inch. Horizontal. $t = \frac{H R}{12 F_v}$ ‡ 10-ft. flood.	Thrusts in Arches, in Pounds per Square Inch. Vertical. $s = \frac{P_n}{F_b}$ ¶ Full pond.	Thrusts in Arches, in Pounds per Square Inch. Vertical. $s = \frac{P_n}{F_b}$ ¶ 10-ft. flood.
(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
0.000	1.414	1.000	54.40	13.45	26.2	291.0	817	98	275	11.1	26.2
0.000	1.414	1.000	54.70	15.26	26.9	345.0	934	108	279	12.8	28.8
0.487	1.345	0.896	56.95	1.93	28.2	36.7	76	90	187	13.4	28.4
0.487	1.230	0.718	57.60	2.06	29.8	14.5	28	33.7	65	13.5	27.5
0.487	1.161	0.600	58.15	2.18	31.1	0.7	23	1.5	51	14.1	32.0
0.487	1.108	0.414	58.60	2.28	32.1	10.9	20	23.3	48	14.7	33.8
0.487	1.068	0.306	59.00	2.48	32.8	9.3	15	18.4	30	15.3	30.0
0.010	1.046	0.277	59.30	2.57	33.2	30.3	54	58.3	104	16.4	29.1
0.010	1.040	0.270	60.05	14.20	40.7	332.7	566	117	199	19.9	32.0
0.010	1.035	0.250	61.15	15.25	45.9	379.2	611	127	204	23.4	35.0
0.010	1.030	0.237	62.12	16.98	50.1	482.4	691	147	211	27.6	39.4
0.010	1.027	0.228	63.05	18.55	55.3	563.5	770	160	218	30.6	42.1
0.010	1.022	0.200	63.87	19.85	57.8	636.5	839	170	225	35.1	46.5
0.010	1.017	0.171	64.60	21.10	62.3	705.3	908	180	231	38.3	49.3
0.010	1.014	0.150	65.32	22.45	65.7	774.9	972	188	237	42.0	53.3
0.010	1.011	0.130	65.87	22.90	69.2	841.0	1 036	202	248	45.3	56.4
0.010	1.007	0.110	66.36	24.68	72.3	909.8	1 106	204	248	48.2	59.5
0.010	1.005	0.089	66.74	25.40	76.3	975.0	1 171	213	257	50.8	61.7
0.010	1.003	0.066	67.08	26.70	79.6	1 043.3	1 234	218	258	53.3	63.5
0.010	1.002	0.044	67.33	28.36	81.6	1 113.8	1 304	220	258	56.7	66.8
0.010	1.001	0.044	67.52	29.50	85.0	1 188.3	1 380	226	263	58.8	68.6
0.010	1.001	0.044	67.67	29.50	87.6	1 264.3	1 455	241	278	61.5	71.0
0.0022	1.000	0.000	67.75	62.00	87.6	2 883.0	3 125	262	285	69.0	78.8
0.058	1.0016	0.0574	67.70	13.60	87.0	482.9	487	200	202	71.5	81.0
0.058	1.0135	0.1648	67.46	13.60	85.5	157.0	117	65	48.3	73.7	93.7
0.058	1.0374	0.2720	67.00	13.30	84.0	148.0	107	62	44.9	75.4	86.2
0.058	1.075	0.3940	66.28	13.10	81.6	164.0	116	69	48.9	77.6	89.0
0.058	1.128	0.5223	65.40	12.80	78.5	157.0	99	67	42.1	79.6	92.2
0.058	1.201	0.6657	64.32	12.10	74.3	175.0	110	78	48.8	83.5	96.3
0.058	1.300	0.8309	63.07	11.50	69.9	205.0	131	94	59.8	85.1	100.4
0.058	1.434	1.0282	61.60	10.70	64.4	259.0	160	124	75.7	88.3	106.0

† Component of water pressure on down-stream face, due to 8 ft. head on lower dam subtracted.

‡ At center of sections.

¶ At base of sections.

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t = the unit pressure, per square inch, on the horizontal arch at the center of the section;

s = the unit pressure, per square inch, on the vertical arch at the base of the section;

then

$$H = 2\,T \sin.\frac{\psi}{2} = \left[p_w - (P_n + P_{n-1}) \sin.\frac{\theta}{2}\right] \sec.\, i + \Delta\, G \tan.\, i. \quad (3)$$

But, by the dimensions of the block, $\sin.\frac{\psi}{2} = \frac{1}{2}$ in. $\div R$, in inches; therefore,

$$H = 2\,T\,\frac{\frac{1}{2}}{R} \quad \text{.......................} \quad (4)$$

or

$$T = H\,R \quad \text{.......................} \quad (5)$$

and

$$t = \frac{T}{F} = \frac{H\,R}{F} \quad \text{.......................} \quad (6)$$

and

$$s = \frac{P_n}{F_b} \quad \text{.......................} \quad (7)$$

For R and F, in feet, Equation 6 becomes

$$t = \frac{H\,R}{12\,F} \quad \text{.......................} \quad (8)$$

Table 10 presents the elements of this computation for a full pond and for a 10-ft. flood.

Comparing the stresses induced for the two cases, Columns 20 to 23, the interesting fact is discovered that in this structure the unit stresses in the horizontal arches of the torus base—the weakest spot, apparently, if the design be judged by inspection simply—are less for the case of a flood than for that of a full pond, and, in spite of the apparently thin section at the toe, the maximum stress is only 124 lb. per sq. in., while the maximum anywhere in the structure, under assumed conditions far beyond any possible contingency, is less than 285 lb. per sq. in. Using the radius of the center of gravity of the cylindrical block, which is approximately 63 ft., the maximum unit stress in the dam is seen to be $\frac{63.00}{67.75} \times 285 = 265$ lb. per sq. in., or 19.08 tons per sq. ft.

Owing to the thicker section of the dam as it approaches the abutments the corresponding maximum pressures on the rock are: for the east abutment, 247 lb. per sq. in., or 17.8 tons per sq. ft.; and, for the west abutment, 211 lb. per sq. in., or 15.2 tons per sq. ft. By way of comparison, it may be recalled that the pressures on the foundations of the Rookery Building, in Chicago, and those of the old Brooklyn Bridge are given as 400 lb. per sq. in., or 28.8 tons per sq. ft., while the concrete and low-grade rubble base of the Washington Monument is subjected to loads of 525 lb. per sq. in., or 37.8 tons per sq. ft. There are also a number of unreinforced concrete-arch bridges abroad which have been standing several years, in which stresses greater than

300 lb. per sq. in. either exist continuously or occur frequently, aside from those due to temperature changes and rib shortening. Mr. Williams.

The stresses near the crest are amply provided for by the crest casting and the steel channels in that portion of the structure.

Having now considered the conditions of full pond and flood, it remains to enquire as to the stresses in certain parts of the structure at low water, and when the pond is empty, should the latter condition ever occur after the completion of the work. Examining the horizontal thrusts due to the weight of the dam, it was found that for Sections 5 to 8, inclusive, the outward thrusts are decreasing downward, whence, in an ordinary dome, tension would occur in this region, and should in that case be resisted. Any yielding in the haunches, however, must be accompanied by a lowering of the crest at the center, and, because of the rigidity of the abutments in this case, which prevents spreading along the chord of the dam, any lowering of the crest will be resisted by the hyperbolic arch formed along the vertical plane through the crest, and, consequently, the hoop usually supplied to a dome at the so-called joint of rupture is not needed here, although, to relieve the small tensions which might occur while the vertical arch resistance was developing, a hoop of 4 by $\frac{3}{4}$-in. steel was provided at the haunch.

At the top of the torus base, similarly, tension would occur with the pond empty, were it not that a system of piers introduced under the heel of the dam acts as a support for the upper masonry at such times. These piers have no bond with the body of the dam, which is free to move away from them when loaded, but they act simply as wedges to keep the structure erect when there is no pressure on the back, and prevent tensile stresses at the top of the torus base.

As already stated, in the design of the Six-Mile Creek Dam, an attempt was made to eliminate, as far as possible, the influence of the beam or cantilever action which plays so extensive a part in the stresses of such curved dams as the one described by the authors, and the Zola and Sweetwater Dams, and even the more rationally designed Bear Valley Dam. To discover how successfully this has been accomplished, the midsection of the dam under a 10-ft. flood was subjected to an analysis similar to that presented for the Lake Cheesman Dam. A vertical slice of the dam, 1 ft. in thickness circumferentially, was taken, and, to simplify computations, rectified by projection upon a vertical tangent to the cylindrical portion, and this was divided into six sections 15 ft. in height. The average moment of inertia and the resulting deflections for each section were computed as for a beam, and then the average deflection of each section as an arch. The results are presented in Table 11, wherein, adopting the authors' notation:

a = height of section = 15 ft.;

D = the deflection;

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TABLE 11.—Comparison of Arch and Beam Stresses in the Six-Mile Creek Dam.

	For Beam.								For Arch.								Percentage of Load Carried by:	
	Elevations.		Thickness, in feet.															
Section.	Top.	Bottom.	Top.	Bottom.	Load, X, in pounds.	Moment of inertia, I.	$\frac{a^3}{6EI}$	$E\,D$, for beam.	Elevation.	Radius, R, in feet.	Area of Ring 1 ft. thick, A, in square feet.	α	$L = \pi R \frac{4\alpha}{180°}$, in feet.	T, in pounds per square foot.	$E\,D = \frac{T\,L \cot.\,\alpha}{2\,A}$.	$\frac{E\,D \text{ for Beam}}{E\,D \text{ for Arch}} = \frac{D \text{ for Beam.}}{D \text{ for Arch.}}$	Arch.	Beam.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	260	245	1.0	3.45	16 400	1.20	$\frac{468}{E}$	883 820 000	252.5	55.93	2.22	26° 48'	104.65	27 200	1 270 000	695	99.85	0.15
2	245	230	3.45	4.93	30 400	6.32	$\frac{89}{E}$	664 800 000	237.5	60.33	4.18	24° 07'	101.61	30 200	822 000	808	99.91	0.09
3	230	215	4.93	6.19	44 400	14.51	$\frac{38.8}{E}$	469 950 000	222.5	62.59	5.56	22° 59'	100.42	33 850	721 000	652	99.85	0.15
4	215	200	6.19	7.25	58 400	25.35	$\frac{22.2}{E}$	298 170 000	207.5	63.61	6.72	22° 31'	100.00	37 100	666 000	448	99.77	0.23
5	200	185	7.25	7.75	68 300	35.20	$\frac{16}{E}$	154 900 000	192.5	63.81	7.50	22° 25'	99.86	40 100	647 000	239	99.54	0.46
6	185	170	7.75	5.70	71 100	25.93	$\frac{21.7}{E}$	46 505 000	177.5	62.63	6.72	22° 58'	100.45	10 380	188 000	254	99.61	0.39

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$I =$ the average moment of inertia for the section;
$E =$ the modulus of elasticity;
$X =$ the horizontal component of load on the section;
$T =$ the thrust of the horizontal arch, the average value for a layer 1 ft. thick being used;
$L =$ length of the arc of the arch;
$A =$ area of arch layer 1 ft. thick;
$\alpha =$ one-quarter arc of arch before loading.

From Columns 18 and 19, the loads carried by the two systems of forces being inversely as the deflections under the same load, it is seen that at no point above Elevation 185, which is only 15 ft. above the base, does the beam carry one-half of 1% of the load, and it is also apparent that at half this distance above the base the beam cannot carry as much as 1 per cent.

Therefore, it may be concluded that the purpose of the designer, in this respect, has been accomplished, and that the stresses presented in Table 10 represent fairly the conditions in the structure.

At the west abutment, where, as already stated, the rock did not rise to the height of the crest, the thrust in the upper portion of the dam was taken up by a concrete abutment rising to Elevation 270, beyond which the dam was continued into the hillside as a series of elliptical half cylinders, the section of which at 45° was a circle, and which were inclined at 45° and cut off 10 ft. above the crest of the dam by a horizontal plane, the cylinders being supported by wedge-shaped piers under their springings.

As this part of the design has no bearing on the case of the Cheesman Dam, it will not be discussed further at this time.

Material and Construction.—The questions of material and construction have no particular bearing upon the Cheesman Dam, wherein the material was unquestionably the most desirable and best that could have been used, but, as considerable interest has been manifested in these matters, and as a description of the Six-Mile Creek Dam would be incomplete without them, they will be added here.

The body of the Six-Mile Creek Dam is of concrete composed of 1 part Alsen's imported Portland cement, 2 parts creek sand, 2 parts creek gravel and 2 parts broken stone from drift boulders, crushed to pass a 4-in. ring or less.

The voids in the sand amounted to about 42% of its volume. Mortar briquettes, 2 of sand to 1 of cement, 7 days old, indicated that it had a strength in tension equal to about two-thirds of that of standard sand.

The creek gravel was ordinary drift mixed with fragments of the shale rock of the region. Where the latter appeared as flat stones they were broken up or raked out.

The crushed stone contained about 15% of selected shale from the

Mr. Williams. excavation in the rock walls, the remainder being field boulders. Flat stones were rejected both before and after crushing, as far as a reasonably close inspection discovered them.

The faces of the dam were of a single course of vitrified paving brick laid in a mortar of 1 part Alsen's cement, 1 part creek sand and 1 part crusher dust, and were anchored into the body by bent steel anchors, $\frac{1}{2}$ by $\frac{1}{8}$ by 7-in., turned up $\frac{1}{2}$ in. at each end, placed at every fifth brick in every fifth course. On the up-stream face of the torus base the bricks were laid with the flat exposed, elsewhere with the edge exposed.

The brick used was that known as Catskill block, 3 by 4 by 9 in., a very thoroughly vitrified shale brick, weighing 144.4 lb. per cu. ft. They were generally burned so highly as to be distorted considerably by the heat. Four samples, immersed in pails of water for four months, increased in weight less than one-tenth of 1%, and, when tested endwise in compression, failed by splitting lengthwise with a sharp report, at pressures varying from 2 300 to 4 600 lb. per sq. in.

Next inside the brick is a 3-in. mortar face of the same mixture as that used for the joints in the brickwork, which was laid at the same time as the concrete body, being separated therefrom by a plate of iron, until both were placed, when the iron was withdrawn and all were rammed together. Within this mortar face, and as close to the brick as convenient, *i. e.*, about 1 in. away, were set, above Elevation 185, bands of 3 by $\frac{3}{16}$-in. steel extending around the structure every 4 ft. in height, and united through the dam every 4 ft. horizontally by steel rods, $\frac{5}{8}$ in. in diameter, with a nut at each side of the bands. At Elevation 185, on the up-stream side, a band of 4 by $\frac{3}{4}$-in. steel was used and connected to the opposite 3 by $\frac{3}{16}$-in. band in a similar manner, to provide for possible tensions from pier to pier when the pond was empty. Over this steel skeleton, which was held in place by the horizontal rods extending into the brick faces, there was laid or hung a netting of crimped $\frac{3}{16}$-in. longitudinal and $\frac{1}{8}$-in. vertical wire of 4-in. mesh, extending from abutment to abutment on each face, and lapped one mesh and wired together at the horizontal joints of the sheets. All iron and steel was grouted carefully by dipping it in a trough as soon as it came on the work and before it had time to rust, and the bands and netting were placed as close to the outer faces as possible, their purpose being to distribute the stresses due to temperature changes and thereby prevent local cracks. The mortar and concrete were mixed in a Ransome mixer located about 150 ft. up stream from the dam, and the material was placed very wet. Into the body of the concrete were forced one-man stones as each layer was put in, they being carefully set with bed planes normal to the line of thrust, and were left projecting about half their height when a section was completed.

FIG. 1.—PREPARING THE FOUNDATIONS FOR THE SIX-MILE CREEK DAM.

FIG. 2.—UP-STREAM VIEW OF EAST END OF SIX-MILE CREEK DAM.

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The brick walls were first laid up to a height of about 4 ft., No. 10 steel wires being bedded in every fifth course on the up-stream side, and, after setting about two days, the concrete was placed between them, they making the forms after Elevation 185 was reached. No deformation of the walls was detected in any part of the work although braces on the up-stream side, used at first, were dispensed with entirely as the work progressed, and the down-stream side was left entirely unsupported all the time. Fig. 2, Plate XXXIX, shows clearly the method of construction and the appearance of the work.

The foundations were carried down to sound rock, usually from 5 to 6 ft., but in one case, for a short distance, to 18 ft., below the bed of the stream. The longitudinal seams in the bottom were followed by 2 or 3-in. drill holes for from 4 to 6 ft. below the bottom of the foundations, and the holes filled with plastic clay well rammed. Fig. 1, Plate XL, shows the end sections of the torus base completed, and excavation in progress for the middle.

Handling of the Stream.—As it was impossible to divert the stream from the gorge, it was necessary to provide for carrying it through the work, and the design contemplated the erection of the portions of the torus base at the abutments and in front of the piers, leaving the intermediate spaces open, but making all excavations, the stream being diverted from side to side during the work. Arches were to be sprung across the openings thus left, and the dam completed above Elevation 185, leaving passages through the base of sufficient capacity to deliver an ordinary flood. A permanent cast-iron culvert, 5 ft. in diameter, was also provided through the base and controlled by a gate. After the upper portions of the dam were completed, the openings through the base were to be filled, one at a time, at low water, the culvert then being able to carry the flow. Fig. 1, Plate XLI, looking up stream, shows the base at the west end of the dam, and the low-water flow of the creek passing through the opening there while the center of the base is building.

The Dam as Built.—When it became noised abroad that a dam 90 ft. high and but 8 ft. thick at the base was to be built only two miles above Ithaca, to form a lake of 60 acres area, many people immediately saw visions of a Johnstown flood, and protests began to appear in the public prints. The plans, meanwhile, had been referred to four prominent members of this Society, the first of whom withdrew without making any report either favorable or unfavorable, and the other three, fully cognizant of all the conditions, including the action of the first, reported an unqualified approval. In due time bids were called for, and six bidders submitted proposals. Of these bidders, four were experienced engineers and contractors, three being members of this Society. Not one of them, after examination of the plans, specifications and location, expressed any doubt as to the stability of the struct-

Mr. Williams. ure. The tenders were received on unit prices for the several kinds of work involved, and the gross bids, exclusive of cement, which was furnished by the Water Company, based on the Engineer's estimates of quantities, were as follows: $63 365; $55 795; $44 280; $38 957; $35 360, and $34 488.

After work was begun, discussion of the structure continued, and a few so-called engineers, who had never seen the plans, expressed themselves in condemnation of the structure. Several others, after examining plans and location, expressed approval, and others still, perhaps more discreet than either, said nothing. The citizens of Ithaca invoked the aid of the State Engineer, the State Flood Commission, and, finally, the State Health Commissioner, and injunction proceedings were threatened, but, apparently for lack of the necessary kind of engineering advice, were never brought. The breaking out of a typhoid epidemic, due to the water supply, but most probably not, as charged, to the infection of the water by work on this structure, raised public excitement to such a pitch that an almost unanimous vote in favor of municipal ownership was taken, although a similar proposition had been defeated less than a year before; and the State Health Commissioner announced at a mass meeting that his consulting engineers had declined to approve the design of the dam; but, he afterward stated, over his signature, that no official report had been made to him on the subject.

In consequence of all this, the company decided to stop the construction of the dam at a height of 30 ft., and, accordingly, it was built on the original lines to Elevation 193 and finished at Elevation 201, with a crest 60 ft. in radius overhanging on the down-stream side and having a 45° up-stream slope, as shown in broken lines in Fig. 30, and photographically in Fig. 2, Plate XL, and Fig. 2, Plate XLI, the former being a view of the up-stream face of the dam at the east end, and the latter the completed dam as it appears from the west abutment.

A flood of 3 ft. depth above the crest went over the dam on August 31st, 1903, before the runway used in its construction had been removed. By descending a ladder at each end of the dam, it was possible to look under the sheet and observe the face of the dam. A single leak, supplying a jet about half the size of a lead pencil, appeared about 8 ft. above the base. At this time the up-stream face of the dam, at the base, where the brick was laid up against forms, had not been pointed. Since pointing, neither this nor any other visible leaks have appeared.

A Test of the Brick Facing Arch.—During the construction, when the concrete was completed to Elevation 185 and the brick walls were up about 4 ft. higher, ready to receive the concrete filling, the work being in a similar condition to that shown by Fig. 2, Plate XXXIX, the brick arch was subjected to an interesting test.

FIG. 1.—SECTION OF BASE OF SIX-MILE CREEK DAM.

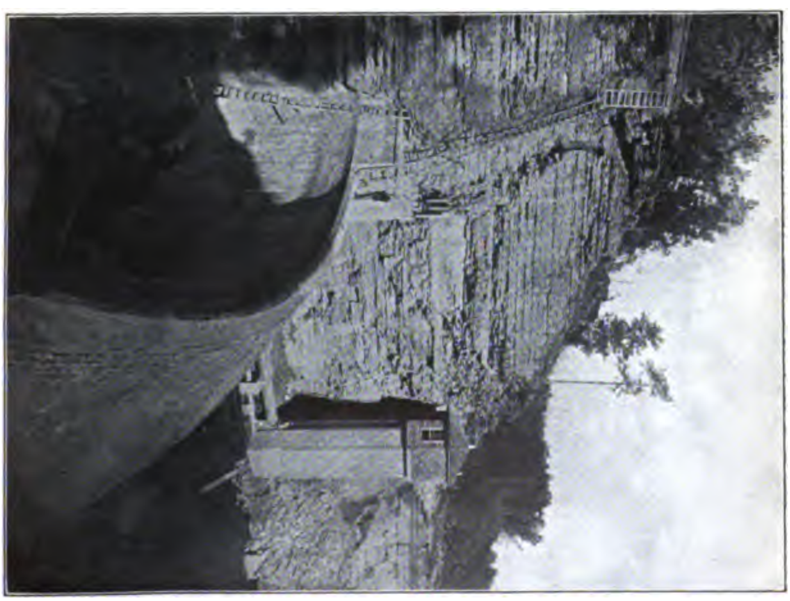

FIG. 2.—THE SIX-MILE CREEK DAM.

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TABLE 12.—Distribution of Stresses and Arch Thrusts in Brick-Facing Arch.

Height 52 in., Thickness 4 in., Span 90 ft., Radius of Extrados 50.08 ft.

Position.	Deflections of Arch Crown.		Water pressure, in pounds per square inch.	Part of Total Load Carried by Arch and Arch Thrusts.									
	Arch carrying entire load.	Vertical beam carrying entire load.		At crown. Percentage of load.	At crown. Thrust, in pounds per square inch.	At $\frac{4}{10}$ span.* Percentage of load.	At $\frac{4}{10}$ span.* Thrust, in pounds per square inch.	At $\frac{3}{10}$ span. Percentage of load.	At $\frac{3}{10}$ span. Thrust, in pounds per square inch.	At $\frac{2}{10}$ span. Percentage of load.	At $\frac{2}{10}$ span. Thrust, in pounds per square inch.	At $\frac{1}{10}$ span. Percentage of load.	At $\frac{1}{10}$ span. Thrust, in pounds per square inch.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
At top	$\frac{78\,000}{E}$	$\frac{185\,500}{E}$	0.346	63.5	38	68.5	41	74.8	45	81.8	49	89.7	54
18 in. below top	$\frac{185\,000}{E}$	$\frac{94\,200}{E}$	0.82	38.8	48	38.9	55	45.8	65	56.2	80	71.9	102
At mid-height	$\frac{291\,000}{E}$	$\frac{51\,800}{E}$	1.28	15.12	34	18.18	41	22.9	51	30.8	69	47.2	106
18 in. above bottom	$\frac{398\,500}{E}$	$\frac{15\,950}{E}$	1.75	3.84	12	4.76	14	6.25	19	9.09	28	16.67	51
At bottom	$\frac{504\,000}{E}$	0	2.22	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0

* Fractional spans measured from abutment.

Mr. Williams. The brick walls, 4 in. thick, had been carried to a height of 52 in. above the top of the concrete on June 19th, 1903. The down-stream wall had all been laid within ten days, having been finished less than a week, and the up-stream wall laid within five days, a stretch of about 6 ft. near the east end, where the runway had been located, having been finished that afternoon. A heavy rain occurred the next day, and the water rose to such a height during the night as to overflow the dam to a depth of 0.8 ft. It began going over about 3 A. M., and was still overflowing at 9:30 A. M., the maximum height probably lasting for 2 or 3 hours.

As the water rose it apparently dissolved the mortar in the short portion of the wall, about 6 ft. long, laid the day before, and broke through there before it went over the top, for the down-stream wall opposite this place was bulged, for about 10 ft., from nothing to about 1¼ in., the maximum being at about two-thirds of its height; and the up-stream brick wall, near the opening thus formed, was tipped over up stream, falling to the bottom of the pond above the dam, thus indicating that a powerful stream of water had recoiled from the lower wall against the concave side of the then unsupported upper wall, and wrecked it. At the far end of the dam, the up-stream wall fell over upon the concrete of the base, showing that the recoil had spent itself before reaching that part of the work.

In spite of having received what must have been a very severe shock, this brick arch, 52 in. high, 4 in. thick, and of 90 ft. clear span, with an up-stream radius of 58 ft. 1 in., stood throughout the flood, and, when the water subsided to a level with the top, no leaks appeared through the wall, except where openings had been left for the insertion of tie rods, and the wall was used, as originally intended, without any repairs or alterations.

TABLE 13.—BEAM STRESSES IN BRICK-FACING ARCH.

Position from abutment.	Part of Total Pressure Carried by Beam 1 In. Wide.		Distance of center of pressure above base, in inches.	Moment at base, in inch-pounds.	Maximum Stresses on Horizontal Mortar Joints, in Pounds per Square Inch.		
	Percentage.	Pounds.			Tension.	Compression.	Shear.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crown.....	87.3	58.32	17.2	1 004	372	380	14.5
$\frac{4}{10}$ span....	85.2	56.89	16.9	962	357	365	14.2
$\frac{3}{10}$ span....	82.1	54.81	16.5	905	335	343	13.7
$\frac{2}{10}$ span....	77.2	51.56	15.8	814	301	309	12.9
$\frac{1}{10}$ span....	68.0	45.45	14.2	645	238	246	11.4

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The stresses set up in this thin arch have been examined by a process similar to that used by the authors, and the results are presented in Tables 12 and 13. One point, not mentioned before, is to be noted, *viz.*, that, in considering a section of a dam as a beam, its action differs from that of an ordinary horizontal beam, since it does not begin to deflect until the load becomes sufficiently great to overcome the moment of the weight of the structure itself about some point in its base; and, if upon a yielding foundation, this point will not be the toe, as is usually incorrectly assumed in dealing with the overturning of gravity sections. Allowance for the weight of the material has been made in the case of this brick arch, but was omitted in the analysis of the Six-Mile Creek Dam, and, apparently, in that of the Lake Cheesman Dam. In the former, by reason of its peculiar profile, it appears that the moment of the weight might become a considerable factor, and while not affecting greatly the deflections in the upper portions, might, to an appreciable degree, reduce that at the bottom, and hence the total deflections of the structure as a beam. Enquiring into this matter, the total weight of a slice of the dam 1 ft. wide is found to be, by Column 11, Table 10, $12 \times 5863.5 = 70\,362$ lb.; and, by inspection of the profile, it appears that the center of gravity will be somewhere within 5 ft. of the point of rotation at the base. The moment of the weight of the structure against overturning, therefore, is less than 350 000 ft.-lb. The total water pressure on the 1-ft. slice of the dam under a 10-ft. flood is $50 \times 62.4 \times 90 = 280\,800$ lb., and the point of application of its resultant is approximately 32.5 ft. above the base. The overturning moment, therefore, is $32.5 \times 280\,800 = 9\,126\,000$ ft-lb. The introduction of the weight factor into the computations of Table 11, consequently, would reduce the bending moment for the beam section less than $\frac{280\,800}{9\,126\,000} = 3.07\,\%$, an amount too small to affect to an important degree the results there set forth.

The values of the stresses given for the brick arch in Table 13 are computed on the usual assumption that the modulus of elasticity of the brick and the mortar joints is the same in compression as in tension. The tensions represent, not only the tensile strength of the mortar in itself, but also its adhesion to the brick surfaces, and the same is to be said for the shears. Tests by briquettes made of 2 parts of creek sand to 1 of cement, with 17% of water, when one week old, gave 270 to 300 lb. per sq. in. tensile breaking stress, and at two weeks 375 to 400 lb. The replacement of 1 part of the sand by 1 part of the crusher screenings probably increased the strength of the mortar somewhat, so that a tensile stress of 372 lb. is within the probable strength limit of the mortar ten days old.

It is noticeable, in such a long and slender arch as is represented

Mr. Williams. by the brick wall, that the arch deflections at the crown would be much greater than the deflections as a beam, and the values for the portions of the loads taken by the two systems are to be looked upon as limiting approximations. The beam cannot be expected to carry more nor the arch less than these quantities. It is noticeable, however, that the thrust of the arch increases toward the abutments by reason of its deflection becoming less. As the radius, thickness and water pressure are unchanged from point to point, this increase can only be provided for or resisted by the absorption by the arch of some of the stress credited to the beam, or by the setting up of shearing stresses of varying amounts, along horizontal planes. It follows that any complete analysis of the stresses in this arch, or any one in which the beam action is an important factor, must involve the consideration, not only of these, but also of secondary stresses set up by the action at right angles of primary stresses of opposite signs, all of which makes a very complicated problem.

In the Six-Mile Creek Dam no such uncertainty exists, as the beam action is practically eliminated, and the only secondary stresses to be considered are those due to rib shortening and temperature, the former of which cannot be nearly as serious as in the case of an arch bridge, with its necessary bending moments under both dead and live loads, and the latter of which is of less consequence than in such a structure, because the range of temperature is necessarily less.

It appears, then, that in the design of the Six-Mile Creek Dam there is presented a structure, which, for all practical considerations, acts wholly as an arch under uniform normal pressures, the equilibrium curve for which condition coincides with the center line of the section, and this seems to prove it to be as near an approach to the ideal as engineers are usually able to accomplish in structures of its magnitude.

The work was executed, under the speaker's personal supervision, by Messrs. Tucker and Vinton, Inc., of New York City, who used every endeavor to produce a most creditable structure. The speaker was assisted upon the construction by S. C. Hulse, Jun. Am. Soc. C. E., and Mr. Weston E. Fuller, whose care and interest in the work also merit commendation.

Vol. XXX. SEPTEMBER, 1904. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE COLLAPSE OF A BUILDING DURING CONSTRUCTION.

Discussion.*

By H. de B. Parsons, M. Am. Soc. C. E.

H. de B. Parsons, M. Am. Soc. C. E. (by letter).—The writer was retained by the District Attorney for the purpose of reporting on the cause of the collapse of the Hotel Darlington. Therefore, in the paper, he confined himself to a rehearsal of the facts as found, coupled with his opinion of the cause of the collapse, as contained in his report. Mr. Parsons.

The outside walls along the edge of the lot were continuous from the foundation, with the exception of a portion of the rear wall, which was carried on girders located at the ground-floor level. The walls around the light wells were curtain walls.

The column schedule given in Table 1 does not refer to any particular column. Table 2 gives the sizes of Columns 2, 35 and 36.

The holes in the girders and beams had no regular size, but varied from $\frac{13}{16}$ in. to 1 in. in diameter. It makes very little difference what the exact size of the hole is, as long as it is larger than the bolt, when the effect of lateral stiffness is considered. In every case the bolts were slack in their holes.

The writer has no knowledge of the estimated weights for live load or for dead load which were used by the architect, or by the builder, in calculating the strength of the building.

* Continued from August, 1904, *Proceedings*. See April, 1904, *Proceedings*, for paper on this subject by H. de B. Parsons, M. Am. Soc. C. E.

Mr. Parsons. TABLE 2.—SIZES OF COLUMNS 2, 35 AND 36 OF THE HOTEL DARLINGTON.

Mark.	Column 2.	Column 35.	Column 36.
A	9 x ¾ in.	9 x 1 in.	9 x 1¼ in.
B	9 x ¾ "	9 x 1 "	9 x 1 "
C	6 x ¾ "	9 x ¾ "	9 x 1 "
D	6 x ¾ "	8 x ¾ "	8 x 1 "
E	6 x ¾ "	8 x ¾ "	8 x 1 "
F	6 x ¾ "	7 x ¾ "	7 x 1 "
G	6 x ¾ "	7 x ¾ "	7 x 1 "
H	6 x ¾ "	6 x ¾ "	6 x 1 "
J	6 x ¾ "	6 x ¾ "	6 x ¾ "
K	6 x ¾ "	6 x ¾ "	6 x ¾ "
L	6 x ¾ "	6 x ¾ "	6 x ¾ "
M	6 x ¾ "	6 x ¾ "	6 x ¾ "
N	6 x ¾ "	6 x ¾ "	6 x ¾ "
O	....	6 x ¾ "	6 x ¾ "

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXX. No. 8.

OCTOBER, 1904.

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1904.

Vol. XXX. OCTOBER, 1904. No. 8.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

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MINUTES OF MEETINGS.

OF THE SOCIETY.

REPORT IN FULL OF THE THIRTY-SIXTH ANNUAL CONVENTION AT ST. LOUIS, MO., OCTOBER 3d, 1904.

2.30 P. M.—The First Session of the Convention was held in the Hall of Congresses, Administration Building, Louisiana Purchase Exposition.

The President delivered the Annual Address.

At the conclusion of the address the Business Meeting was held, President Hermany in the chair; Chas. Warren Hunt, Secretary.

General William P. Craighill, Past-President, Am. Soc. C. E.—Mr. President, if it is in order, I would like to offer a resolution of a few lines, which seems to me to be appropriate at this time, and I hope it will be seconded and unanimously adopted.

"*Whereas*, The International Congress of Engineers, now in session in such successful progress in this city, is the result of the suggestion and management of the American Society of Civil Engineers, through the agency of its committee, of which Henry S. Haines is chairman, and Chas. Warren Hunt is secretary;

Resolved, That the earnest thanks of the American Society of Civil Engineers are hereby cordially given to our committee for the able, faithful and excellent way in which its duties have been performed, in the general arrangements for the Congress, as well as the many and varied details involved in the outcome now before us."

Seconded by G. A. Marr, M. Am. Soc. C. E.

General Craighill.—It will be observed, Mr. President, that in the last few words I refer to the outcome now before us. One prominent feature of that outcome which is most interesting to me, and must and will be, I suppose, interesting to every person present, is the fact that we have on the same platform here the President of the British Institution of Civil Engineers as our distinguished guest, Sir William White, and the President of the American Society of Civil Engineers. That is enough to justify everything that has been done. (Applause.)

But I wish also to call attention to a fact, which strikes me as an officer of the army, as well as a member of the Society, and that is the fact that we see around us, not only our own star-spangled banner, but the ensign of Great Britain, floating over our heads, and many flags of other Nations. But there is another outcome, Mr. President, to which I want to call special attention, and that is what appeals to me as I face these men before me; it is the bringing together of men of brains, men of energy, and men of activity, who know how to design, and how to execute. And, last but not least, I want to call attention to the presence of the ladies. Without them we can do nothing. (Applause.)

The President.—Are there any further remarks? If not, the chair will put the resolution.

The resolution was unanimously adopted.

The Secretary.—Mr. President, before some members of the Congress who are not interested in our Business Meeting leave the hall, I should like to make it known that the Engineers' Club of St. Louis has gotten out a very nice World's Fair Souvenir, a number of copies of which are in the Secretary's office, and which can be had upon application there.

It is possible that some of the members of the Board of Direction might leave before the close of this meeting, and I will therefore announce that the meeting of the Board of Direction will be held at 9:30 o'clock to-morrow morning at the office of the Secretary in this building.

Mr. President, the first business to come before this meeting is the question of determining the time and place of holding the next Annual

Convention of the Society. Some time ago the usual form of circular letter was sent to all members, asking them to reply, giving their choice for the time and place of holding the next Annual Convention. I have a report of the answers received, and I beg to report the result as follows:

Total Number of Votes Received for Place, 798.

Distributed as follows:

Cleveland	586	Denver	6
Duluth	52	Washington, D. C	6
Portland, Ore.	35	City of Mexico	4
Boston	15	Mackinac Island	4
New Orleans	11	Philadelphia	4
Pittsburg	9	Seattle	4
New York	8	San Francisco	4
Chicago	7		

The following places received two votes each:

Cape May,
Cincinnati,
Detroit,
Eureka Springs,
Havana,
Lake Champlain,
Louisville,
Quebec,
Saratoga Springs,
Salt Lake City,
Thousand Islands,
Yellowstone Park.

The following places received one vote each:

Alexandria Bay, N. Y.,
Atlantic City,
Baltimore,
Buffalo,
Hot Springs, Va.,
Kansas City,
Lake George,
Los Angeles,
Mackinaw,
Memphis,
Milwaukee,
Minneapolis,
Montreal,
Niagara Falls,
Palm Beach,
Port Arthur,
Saratoga,
Near Portland Cement Industry,
No Choice.

Many duplicate ballots have come in too late for reclassification, and the above list gives all the ballots, in which a number of members have voted twice. In most cases these ballots have been in favor of Cleveland, Duluth, and Portland, and a correct return of all votes would take a number of ballots from other places.

Total Number of Answers Received Suggesting the Time for Next Convention, 798.

	No. of Votes.		No. of Votes.
January	1	Mardi Gras – New Orleans	4
March	1	June to September	1
April	3	June or September	1
May	8	July or August	21
June	618	July to September	1
July	25	September or October	1
August	22	October or August	1
September	25	During Lewis and Clark Exposition, Portland	5
October	27	November to April	1
November	3	Early Summer	1
December	2	First Half of Any Month	1
		No Choice	25

I also have the following letters:

"SEPTEMBER 9TH, 1904.

"TO THE PRESIDENT AND MEMBERS OF THE AMERICAN SOCIETY OF CIVIL "ENGINEERS, IN SESSION AT ST. LOUIS, MO.

"GENTLEMEN: On behalf of the business men of Cleveland, the Board of Directors of the Cleveland Chamber of Commerce extends to your Society a very cordial invitation to hold its next meeting in Cleveland. The directors are confident that your delegates would thoroughly enjoy the time spent in Cleveland, and it is sincerely hoped that the city may be honored by an acceptance of this invitation.

"By order of the Board of Directors of the Cleveland Chamber of Commerce.

"F. A. SCOTT,
"*Secretary.*"

"DULUTH, MINN., SEPT. 8TH, 1904.

"TO THE PRESIDENT AND MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, ST. LOUIS, MISSOURI.

"GENTLEMEN: The Commercial Club of Duluth hereby extends a cordial invitation to your Association to hold your convention for 1905 in our city.

"There are many reasons why we ask for this courtesy from your Association, believing that the Northwest is entitled to the convention of the society, and that it is entirely proper that the third port in volume of business in the United States should be recognized.

"We have a most excellent convention hall which would be furnished free for the use of the convention. We have some of the best hotels in the State with ample accommodations for the delegates. We have an ideal climate at all times and particularly at the time in which the convention will be held.

"Duluth-Superior harbor is one of the finest in the world, containing forty-nine miles harbor frontage, with an area of eleven and three-quarters square miles. About it are located mammoth ore and coal docks, immense grain elevators with a capacity of thirty-five million bushels, a blast furnace, coking plant, shipyard, dry docks, immense lumber mills, etc. The novel Aerial Bridge, nearly 400 ft. span, at a height of 188 ft., is in process of erection across the Duluth Ship Canal, and will be completed this year.

"We believe that Duluth and its surroundings offer many attractions both from the standpoint of engineering and scenic beauty, and we think that the people will want to come and see the Metropolis of the great Northwest.

"We urge you to accept our invitation to come to this City, and are satisfied that if you do, you will want to come again.

"Very respectfully,

"W. A. McGonagle, *President.*

"H. V. Eva, *Secretary.*

"*Convention Committee* { E. L. Millar, W. B. Silvey, E. C. Little."

I also have a letter from H. H. Wadsworth, M. Am. Soc. C. E., enclosing a certified copy of a resolution passed by the Common Council of the City of Duluth, September 6th, 1904, and a letter from the Northern Steamship Co., promising co-operation in the matter of transportation. The Resolution referred to is as follows:

By Alderman Wing:

"*Resolved:* That the attention of the American Society of Civil Engineers is hereby called to the advantages of Duluth as a convention city, it being easily accessible either by boat or rail, with a most delightful summer climate, excellent hotel accommodations, interesting municipal and manufacturing plants and the mines of the Mesaba and Vermilion iron ranges being in close proximity.

"*Resolved, further,* That said Society is hereby extended a cordial invitation to hold their annual session for the year 1905 at this place.

"Alderman Wing moved the adoption of the resolution, and it was declared adopted on the following vote:

"Yeas—Alderman Barnes, Chesney, Harker, McEwen, Moore, Olson, Sang, Schaffer, Tessman, Tischer, Waugh, Wilson, Wing, President Haven—14.

"Nays—None.

"Passed Sept. 6, 1904.

"Approved Sept. 8, 1904."

I also have the following letters from Oregon:

"Salem, September 15th, 1904.

"Charles Warren Hunt, Esq.,

"*Secretary American Society Civil Engineers,*

"New York City, N. Y.

"Dear Sir: I am advised that the American Society of Civil Engineers are to hold their annual convention next month in St. Louis, and that they will at that time select a place for holding the convention for the year 1905. As Executive of this State I desire to extend to your Society a most cordial invitation to meet in Portland, Oregon, at that time. The people of the State are making extensive preparations to celebrate next year the hundredth anniversary of the expedition of Lewis and Clark to the Oregon country. There has been contributed to this end by our own people something near one million dollars, the Congress of the United States has contributed liberally, and our sister States have all been generous in doing whatever is necessary to make the Exposition a success.

"If your Society will meet with us here next year I can assure you that they will be royally received and entertained by people of Oregon, and in addition to that they will enjoy a magnificent climate, and the most beautiful scenery that it has ever been their lot to experience.

"Trusting that you may be able to induce your Society to accept this invitation, I have the honor to remain,

"Yours very respectfully,

"GEO. A. CHAMBERLAIN."

"PORTLAND, SEPT. 12TH, 1904.

"MR. CHARLES WARREN HUNT,

"*Secretary of the American Society of Civil Engineers, No. 220 West Fifty-seventh street, New York, New York.*

"DEAR SIR: As Mayor of the City of Portland, Oregon, I respectfully and earnestly invite you to hold the next meeting of your Association in this City, in 1905, at which time there will be here the Lewis and Clark Centennial and American Pacific Exposition and Oriental Fair.

"It is expected that at this Exposition there will be a magnificent display of the productions of the Pacific States and Territories, with many contributions from the Oriental World.

"To those of you who live in the East, the journey across the Continent will be both interesting and instructive, and without such a journey you can have no adequate idea of the territorial extent and greatness of our country.

"Portland is a city of about 120 000 inhabitants, with a rapidly increasing population. It is situated on the Willamette River, about 100 miles from the Pacific Ocean and 12 miles from the Columbia River, and ocean-going steamships and sailing vessels from all parts of the commercial world visit its harbor.

"I can confidently say that if you will accept this invitation you will find Portland a healthful, prosperous and beautiful city.

"Oregon scenery is diversified with rivers and lakes, hills and vales, fertile valleys and mountains, some of which are covered with perpetual snow, and its grandeur and beauty in these respects are unsurpassed by any country.

"Portland will be pleased to extend you its hospitality.

"Yours very truly,

"GEO. H. WILLIAMS,

"*Mayor.*"

THE PRESIDENT.—What will you do with the report of the Secretary on the question of place and time of holding our next Annual Convention?

ROBERT MOORE, PAST-PRESIDENT, AM. SOC. C. E.—I believe that I neglected to vote in response to the petition of the Board of Direction. I now desire to offer this resolution, to-wit: "That it be the sense of this meeting that the next Annual Convention should be held in the City of Cleveland."

I know that that was very largely the feeling of the members on this subject from the vote taken two years ago, and I think it is now timely that we should hold a meeting in a central State like that, particularly in a city that is so enterprising, beautiful and hospitable as the City of Cleveland. I therefore make that motion.

Motion seconded by Mr. Smith.

J. N. CHESTER, M. AM. SOC. C. E.—With all due respect to commercial points, I believe that the success of our conventions, as well as the pleasure of those who attend, would depend largely upon one thing, and that is a hotel large enough to accommodate the entire convention, and I do not believe the cities that have been mentioned will afford such accommodation. So far as time is concerned, of course the location of the convention should in a measure decide the time, but I would urge that we consider our own convenience and the success of our convention rather than the wishes of some commercial center, or any place that might desire us to come there. I should not even vote unless conditions which I have before mentioned could be met with, and I further urge that we select some place where there is a hotel that can accommodate us and do so comfortably.

G. S. WILLIAMS, M. AM. SOC. C. E.—I would move to amend the motion just presented by striking out all after the enacting clause and substituting, "that the thanks of this society be extended to the cities (or their representatives) of Cleveland, Duluth and Portland, and that the time and place of holding the next Annual Convention be referred to the Board of Direction with power."

Seconded by General Craighill.

MR. MOORE.—It is only fair to the Board of Direction that this meeting should express its preference. The motion I made was simply that it be the sense of this meeting. It is very important that the Board of Direction should know what the real wishes of the members here assembled are, and I do not think we ought to evade that by such an amendment as that proposed by Mr. Williams. I think the meeting should express itself one way or the other. There is a choice of cities and places, any of which will be good, and I do not think there is any reason for dodging it now.

GEO. W. FULLER, M. AM. SOC. C. E.—It seems to me that the result of the letter-ballot ought to have influence in a matter of this kind, because those who have voted have expressed their opinion, and a great many more votes have been received than there are persons present.

A vote on the amendment offered by Mr. Williams was taken and resulted in sixty-five yeas and thirty-four nays, and the motion as amended was adopted.

THE SECRETARY.—The Secretary knows of no further business to come before the meeting.

On motion, the Business Meeting adjourned.

October 19th, 1904.—The meeting was called to order at 8.40 P. M., R. S. Buck, Director, in the chair; Chas. Warren Hunt, Secretary, and present, also, 156 members and 71 guests.

A paper by C. C. Schneider, M. Am. Soc. C. E., entitled "The Structural Design of Buildings," was presented by the author.

The Secretary presented communications on the subject by Messrs. W. B. W. Howe, Charles Worthington, J. R. Worcester and L. J. Johnson.

The subject was discussed orally by Messrs. Henry B. Seaman, Jos. H. O'Brien, Augustus Smith, F. T. Llewellyn, H. P. Macdonald, H. W. Brinckerhoff, Frederick Wilcock, G. H. Blakeley, J. F. O'Rourke, O. E. Hovey, John B. Clermont, V. I. H. Hewes, E. W. Stern and the author.

The Secretary announced that a Special Committee of the Board of Direction canvassed the ballot for membership on October 5th, 1904, and that the following candidates were elected:

AS MEMBERS.

RAYMOND FRANCIS ALMIRALL, New York City.
HUBERT KEENEY BISHOP, Hudson, N. Y.
FRANCIS OGDEN BLACKWELL, New York City.
CHESTER HARVEY CHAMBERLIN, Boyce, La.
THOMAS FENNING CHAPPELL, New York City.
SPENCER COSBY, Manila, Philippine Islands.
WILLIAM GRIFFING FORD, Brooklyn, N. Y.
JAMES FORGIE, New York City.
JAMES LINCOLN FYFE, Chicago, Ill.
EDWARD DANA HARDY, Washington, D. C.
LEONHARD JOHN HOHL, Oroville, Cal.
RUFUS CAMERON HUNT, New York City.
HERMANN KOWER, Berkeley, Cal.
FREDERICK WILLIAM LA FORGE, Fort Terry, N. Y.
OTTO HEINRICH LANG, Dallas, Tex.
WILLIAM DOMINICK LARRABEE, Los Angeles, Cal.
FRANK MILLER, New York City.
EUSEBIUS JOSEPH MOLERA, San Francisco, Cal.
JOHN EDWIN MOORE, Chicago, Ill.
GEORGE HARRISON NEILSON, Braeburn, Pa.

Willard Pope, Walkerville, Ont., Canada.
Wendell Monroe Reed, Roswell, N. Mex.
Louis Davidson Ricketts, Globe, Ariz.
Walter Herbert Sears, Katonah, N. Y.
Harry Randolph Talcott, Cumberland, Md.
Frank Teichman, San Francisco, Cal.
James Wilson Grimes Walker, Charleston, S. C.

As Associate Members.

William Brokaw Bamford, New York City.
Frank Colburn Bowler, Millinocket, Me.
Stephen Pearson Brown, Stratford, Conn.
Newton Albert Kendall Bugbee, Trenton, N. J.
Charles Joseph Carroll, Durango, Mexico.
George Washington Corrigan, Self, Ark.
Jackson Colborn Hitchman, City of Mexico, Mexico.
Stephen Ephraim Kieffer, Berkeley, Cal.
Archibald Angus Macdonald, New York City.
John de Navarre Macomb, Jr., Lawrence, Kans.
Alfred Moyer Meyers, Milwaukee, Wis.
William Pool Parker, Kansas City, Mo.
George Whitfield Pfeiffer, Daiquiri, Cuba.
Louis Adams Robb, Newark, N. J.
William Kerper Runyon, Newark, N. J.
Thomas Bartlett Sears, Lawrence, Kans.
Frank Cummings Shepherd, New York City.
Tyrrell Bradbury Shertzer, New York City.
Chester Wason Smith, Clinton, Mass.
William Freeman Stevenson, New York City.
James Boorman Strong, Niagara Falls, N. Y.
Ralph Cone Taggart, New York City.
Richard Fenwick Thorp, Munaar, Madura Dist., South India.
Austin King Tiernan, Salt Lake City, Utah.
Charles Lucas Wachter, New York City.

As Associate.

James Morton Caird, Troy, N. Y.

The Secretary announced the following deaths:

Alexander Macomb Miller, elected Member June 6th, 1888; died September 14th, 1904.

Van Norman McGee, elected Associate Member, June 6th, 1900; died September 28th, 1904.

Adjourned.

ANNOUNCEMENTS.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

MEETINGS.

Wednesday, November 2d, 1904.—8.30 P. M.—A regular business meeting will be held. "A New Method of Tunneling as Applied to the Construction of the 14 and 15-ft. Bay Ridge Sewer," will be described by James C. Meem, Assoc. M. Am. Soc. C. E., and illustrated with lantern slides.

Wednesday, November 16th, 1904.—8.30 P. M.—At this meeting Walter C. Parmley, M. Am. Soc. C. E., will address the Society and describe the Walworth Sewer, Cleveland, Ohio, illustrating his remarks with lantern slides.

UNIVERSAL EXPOSITION, ST. LOUIS, 1904.

The Society has undertaken to provide for an engineering exhibit, and the establishment of Headquarters for visiting engineers **in the center of the Liberal Arts Building**, and the Board of Direction has appropriated sufficient funds to defray the necessary expense.

This matter is in the hands of the following committee:

Robert Moore, M. Am. Soc. C. E., St. Louis, Mo., *Chairman*.
Edward C. Carter, M. Am. Soc. C. E., Chicago, Ill.
Mordecai T. Endicott, M. Am. Soc. C. E., Washington, D. C.

James L. Frazier.	"	"	Frankfort, Ind.
William Jackson,	"	"	Boston, Mass.
Emil Kuichling,	"	"	New York, N. Y.
J. L. Van Ornum.	"	"	St. Louis, Mo.
John F. Wallace,	"	"	Chicago, Ill.
O. E. Mogensen, *Sec'y*,	"	"	St. Louis, Mo.

PRIVILEGES OF LOCAL SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

The **Boston Society of Civil Engineers** will welcome any member of the American Society of Civil Engineers at its library and reading room, 715 Tremont Temple, Boston, which is open on week days from 9 A. M. to 5 P. M. Members will also be welcome at the meetings, which are held in the same building, on the evenings of the fourth Wednesday in January, and the third Wednesdays of other months, except July and August.

The rooms of the **St. Louis Engineers' Club,** in the business center of St. Louis, will be kept open during the World's Fair season, May 1st to December 1st, 1904, and visiting engineers are cordially invited to use them for mail, telephone service, information, etc.

The courtesies of the **Engineers' Society of Western Pennsylvania** have been extended to members of the American Society of Civil Engineers. The rooms of the Society, 410 Penn Ave., Pittsburg, Pa., are open at all times, and meetings are held as follows, except during July and August. REGULAR SECTION, Third Tuesdays; CHEMICAL SECTION, Thursdays following third Tuesdays; MECHANICAL SECTION, first Tuesdays; STRUCTURAL SECTION, Fourth Tuesdays.

The Western Society of Engineers, Monadnock Block, Chicago, Ill., tenders to members of this Society the use of its rooms and facilities, together with the good offices of its Secretary and of a special committee appointed for that purpose.

The Civil Engineers' Club of Cleveland, Ohio, invites members of this Society to make use of the Club rooms, at any time when in Cleveland. Cards will be furnished on application to the Secretary, Mr. J. C. Beardsley.

SEARCHES IN THE LIBRARY.

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many such searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling, compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

Copies of all lists of references are filed, so that in many cases it is only necessary to make a typewritten copy, which reduces the cost of searches to a minimum.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

LIST OF NOMINEES FOR THE OFFICES TO BE FILLED AT THE ANNUAL ELECTION, JANUARY 18th, 1905.

The following list of nominees for the offices to be filled at the Annual Meeting, January 18th, 1905, received from the Nominating Committee, was presented to the Board of Direction at its meeting on October 6th, 1904. The list has already been mailed to all Corporate Members.

For President, to serve one year:

C. C. Schneider, New York City.

For Vice-Presidents, to serve two years:

M. L. Holman, St. Louis, Mo.
Emil Kuichling, New York City.

For Treasurer, to serve one year:

Joseph M. Knap, New York City.

For Directors, to serve three years:

Morris R. Sherrerd, Newark, N. J., representing District No. 1.
Austin L. Bowman, New York City, representing District No. 1.
Hezekiah Bissell, West Medford, Mass., representing District No. 2.
Edwin A. Fisher, Rochester, N. Y., representing District No. 3.
William B. Landreth, Albany, N. Y., representing District No. 3.
George S. Pierson, Kalamazoo, Mich., representing District No. 5.

ACCESSIONS TO THE LIBRARY.

From September 13th to October 11th, 1904.

DONATIONS.*

MAXWELL'S THEORY AND WIRELESS TELEGRAPHY.

Part I, Maxwell's Theory and Hertzian Oscillations. By H. Poincaré; translated by Frederick K. Vreeland. Part II, The Principles of Wireless Telegraphy; by Frederick K. Vreeland. Cloth, 9 x 6, 11 + 255 pp., illus. New York, McGraw Publishing Company, 1904. $2.

The author states that the object of this book is to give a physical treatment of Maxwell's theory and its applications to some modern electrical problems,—to set forth the fundamental principles which underlie all electrical phenomena, according to Maxwell and his followers, to show how these principles explain the ordinary facts of electricity and optics and to derive from them a practical understanding of the essentials of wireless telegraphy. He states that it has been his purpose in Part II to take up the thread where M. Poincaré dropped it, carrying the line of thought into the practical field of wireless telegraphy, and applying the principles laid down in Part I to the various problems involved; not intending it to be a treatise on wireless telegraphy, the object being rather to deal with principles and to trace the development of the art in its essential features. Where specific cases are cited they are chosen with reference to their fitness to illustrate an idea, and they are treated with a view to emphasize that which is essential and minimize superficial or unimportant details. An alphabetical index of six pages is included.

THE STORY OF AMERICAN COALS.

By William Jasper Nicolls, M. Am. Soc. C. E. Cloth, 8 x 6 in., 396 pp. Philadelphia and London, J. B. Lippincott Company, 1904.

The preface states that the "Story" has been written for those interested in American Coals either as operators, miners, dealers, carriers, or the multitude of consumers—the American people. During years of employment in the coal fields of Pennsylvania, the writer has gathered the material from every available source, and added it to his practical knowledge gained by experience. The simple arrangement of the chapters, beginning with the origin of coal and its development, together with a description of the different routes by which it reaches the consumer, and the various uses to which it is put, is followed by a complete index, so that the book can be used for reference. This second edition has been carefully revised and brought up to date. Since the first was issued the United States has passed from second place in coal production to the head of all other nations. The index covers eighteen pages.

TELEPHONY.

A Manual of the Design, Construction and Operation of Telephone Exchanges. In Six Parts. Part V, The Substation. By Arthur Vaughan Abbott. Cloth, 8 x 6, 17 + 473 pp., illus. New York, McGraw Publishing Company, 1904. $1.50.

The fifth part of Mr. Abbott's work on the Telephone is devoted to the substation. The author says that little that is novel can be claimed for the present presentation. Historical references are of the briefest, as thousands of devices are not even mentioned because, however ingenious, or otherwise, they have for one reason or another failed to meet the test of time and experience. The attempt has been made to present the subject in its practical aspect only, but withal to embrace the point of view of the subscriber as well as that of the telephone manager. The Contents are: Introduction; The Receiver; Telephone Transmitters; Induction Coils and Sub-Station Circuits; Transmission and Current Supply; Signalling Apparatus; Protection; Party Lines; Sub-Station Assemblage; Costs of Installation and Operation.

Gifts have also been received from the following:

Alaska Treadwell Gold Min. Co. 1 pam.
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BY PURCHASE.

The Encyclopedia Americana. Vol. 16. New York, Chicago, The Americana Company.

Proceedings of the Indiana Engineering Society. 20th–24th Annual Reports. Greenfield and Indianapolis, Ind., 1900–04.

Proceedings of the Association of County Surveyors and Civil Engineers of the State of Indiana at its Second Annual Meeting held in Indianapolis, January 17th and 18th, 1882; together with the Constitution and By-Laws, Registered Members, etc. Indianapolis, 1882.

Repertorium der Technischen Journal-Literatur. Herausgegeben in Kaiserlichen Patentamt. Jahrgang 1903. Berlin, Carl Heymanns, 1904.

SUMMARY OF ACCESSIONS.

September 13th to October 11th, 1904.

Donations (including 14 duplicates)	123
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McGEE, VAN NORMAN............. Elected Associate Member, June 6th, 1900; died September 28th, 1904.
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PRATT, WILLIAM ARTHUR...........Elected Member, December 4th, 1895; died September 19th, 1904.
SHIRREFFS, REUBEN.................Elected Member, June 4th, 1890; died August 31st, 1904.

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(September 11th to October 10th, 1904.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1122 Girard St., Philadelphia, Pa.
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(4) *Journal*, Western Soc. of Engrs., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
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(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
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(26) *Electrical Review*, London, England.
(27) *Electrical World and Engineer*, New York City, 10c.
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(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des École Spéciales de Gand*, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Économique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill., 10c.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 50c.
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(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.
(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 410 Penn Ave., Pittsburg, Pa., 50c.
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(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
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(76) *Brick*, Chicago. 10c.
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LIST OF ARTICLES,

Bridge.

Highway Bridges. C. M. Canady. (58) June.
Plate Girder Webs. (12) Sept. 9.
The American Bridge Company's Ambridge Works: The Eye Bar Shop.* (20) Sept. 15.
Design in Relation to the Durability of Bridges. (12) Sept. 16.
An Interesting Draw Bridge Built Mostly of Wood in Forty Working Days.* Martin Schreiber. (72) Sept. 20.
Bridge Renewal Under Unusual Conditions.* (13) Sept. 22.
The Sydney Harbor Bridge, Australia.* (13) Sept. 22.
The Tula and the Chone River Bridges, Mexico.* (14) Sept. 24.
New Bridges at Como Park, St. Paul.* (17) Sept. 24.
Handling Traffic During Bridge Construction in Portland, Oregon.* (17) Sept. 24.
Rebuilding a Wrecked Railroad Bridge at Easton, Pa.* (14) Oct. 1.

Electrical.

The Phipps Power Building.* Gerald E. Flanagan. (58) Aug.
Description of the Electrical Equipment of an Engine Works and Shipyard, with Notes Thereon. H. O. Wraith, Assoc. M. I. E. E. (77) Aug.
A Theoretical Consideration of the Currents Induced in Cable Sheaths, and the Losses Occasioned Thereby. M. B. Field, M. I. E. E. (77) Aug.
Some Uses of the Oscillograph.* D. K. Morris, M. I. E. E., and J. K. Catterson-Smith. (77) Aug.
The Equipment of an Engine Test-House.* R. K. Morcom, M. I. E. E. (77) Aug.
Answers to Questions Relative to High-Tension Transmission (by Committee of Amer. Inst. E. E.) (42) Sept.
Experiments on Sheath Losses of Three-Core Cables.* J. T. Morris. (73) Sept. 2.
Some Recent Long-Distance Submarine Telephone Cables.* F. Breisig. (Abstract of Paper read before the Elektrotechnischer Verein of Berlin.) (73) Sept. 2.
Electricity from Water-Power. A. A. Campbell Swinton. (Paper read before the British Assoc. for the Advancement of Science.) (26) Sept. 2; (73) Sept. 2; (47) Sept. 10; Abstract (12) Sept. 2.
The Puget Sound Power Company.* (12) Serial beginning Sept. 2.
Electric Harbour Cranes.* (26) Sept. 2.
Energy Losses in Magnetising Iron.* W. M. Mordey and A. G. Hansard. (Paper read before the British Assoc. for the Advancement of Science.) (73) Sept. 2; Abstract (12) Sept. 2.
A 100-Mile Transmission Line. Robert Howes. (Abstract of Paper read before the National Elec. Light Assoc. of America.) (73) Sept. 2.
Management of Dynamos. (47) Sept. 3.
The Use of Electricity on the North-Eastern Railway and upon Tyneside.* Charles H. Merz and William McLellan. (Paper read before the British Assoc. for the Advancement of Science.) (12) Sept. 9.
How Message Wires are Guarded on the Continent. M. Petit. (26) Sept. 9.
Experiments for Improving the Construction of Practical Standards for Electrical Measurements. (Abstract of Com. Rept. of the British Assoc. for the Advancement of Science.) (73) Sept. 9.
Series Running of Enclosed Arcs. H. Stafford Hatfield. (73) Sept. 9.
The Installation of Electric Cables.* H. W. Buck. (Paper read before the Electric Club.) (73) Sept. 9.
The Westinghouse Oil Switches for High-Tension Circuits.* (26) Sept. 9.
The Prevention of Accidents in Handling Electrical Apparatus. (26) Sept. 9.
The Hopkinson Test as Applied to Induction Motors.* W. E. Sumpner and R. W. Weekes, M. I. E. E. (Paper read before the British Assoc. for the Advancement of Science.) (26) Sept. 9; Abstract (73) Sept. 9.
North Metropolitan Electric Power Supply and Tramways.* (26) Serial beginning Sept. 9.
The Northern California Power Company's Systems.* (27) Serial beginning Sept. 10.
Storage Battery Exhibit at St. Louis.* (27) Sept. 10.
The Turbo-Electric Power System in Paint Manufacture.* (27) Sept. 10.
The Inspection and Insurance of Electrical Machinery.* Michael Longridge, M. Inst. C. E (47) Sept. 10.
The Design of Dynamos. A. Press. (26) Sept. 16.
The Reaction of the Radiation on a Moving Electron. M. Abraham. (Paper read before the British Assoc. for the Advancement of Science.) (73) Sept. 16.
The Twickenham and Teddington Electricity Undertaking.* Max. J. E. Tilney. (73) Sept. 16.
Raising and Lowering Gear for Arc Lamps.* Donald Smeaton Munro. (26) Sept. 16.
The Magnetic Reluctance of Air Cores in Short Coils.* W. M. Thornton. (73) Sept. 16.
The Propagation of Electric Waves Along Spiral Wires and on an Appliance for Measuring the Length of Waves used in Wireless Telegraphy. J. A. Fleming. (Abstract of Paper read before the British Assoc. for the Advancement of Science.) (73) Sept. 16.
Medium-Span Line Construction.* C. A. Copeland. (Paper read before the Pacific Coast Elec. Transmission Assoc.) (73) Sept. 16.
The New Building for the General Electric Machine Shop and Power House, Lynn.* (14) Serial beginning Sept. 17.

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* Illustrated.

Electrical—(Continued).

The Maintenance Organization on the Bay Counties Power Co.'s Works.* (14) Sept. 17.
Design of Armature Coils.* (27) Sept. 17.
A New System for Securing Secrecy in Wireless Telegraph Signaling.* Emile Guarini. (19) Sept. 17.
The Malcotti Telecryptograph for Telegraphing upon Telephone Lines.* Emile Guarini. (46) Sept. 17.
The Jungner Nickel-Iron Accumulator and its Origin.* M. V. Schoop. (19) Sept. 17.
Hydraulics in Connection with Electric Railway Work. B. E. Morrow. (72) Sept. 20.
Large Bulb Incandescent Electric Lamps as Secondary Standards of Light. J. A. Fleming. (Abstract of Paper read before the British Assoc. for the Advancement of Science.) (29) Sept. 23.
On the Heating of Three-Phase Cable Sheaths.* M. B. Field. (26) Sept. 23.
The Care of Switchboards. (26) Sept. 23.
The Three-Phase 40 000-Volt Power Transmission from Gromo to Nembro (Lombardia).* (26) Sept. 23.
The Willesden Power House of the North Metropolitan Electric Power Supply Co.* (73) Serial beginning Sept. 23.
The Duplex Balancing of Telegraph Cables.* W. Gaye. (73) Serial beginning Sept. 23.
The Starting of the Heyland Single-Phase Induction Motor.* A. Boyd. (73) Sept. 23.
The Electromotive Force of the Clark Cell. (73) Sept. 23.
On the Predetermination of Plunger Electromagnets.* Silvanus P. Thompson. (Paper read before the International Elec. Congress.) (73) Sept. 23.
A 16-Mile 80 000-Volt Experimental Electric Transmission Line. A. C. Balch. (Paper read before the Pacific Coast Elec. Transmission Assoc.) (73) Sept. 23.
Report on High Voltage Transmission Lines (to the St. Ry. Assoc. of the State of N. Y.) (17) Sept. 24.
Testing of Large Capacity Induction Wattmeters. E. W. Gough. (27) Sept. 24.
American Practice in High-Tension Line Construction and Operation. F. A. C. Perrine, M. Am. Inst. Elec. E. (Abstract of Paper presented before the International Elec. Congress.) (13) Sept. 29.
The Puyallup River Water Power Development near Tacoma, Washington.* (13) Sept. 29; (14) Oct. 1; (27) Serial beginning Oct. 1.
Aluminium Electrical Conductors. Roderick J. Parke. (Abstract of Paper read before the Canadian Elec. Assoc.) (26) Sept 30.
A New Electro-Chronograph.* David Owen. (73) Sept. 30.
Electrolytic Receivers in Wireless Telegraphy. Lee de Forest. (3) Oct.
Telpherage.* C. J. Messer. (3) Oct.
Speed Control of Motors.* Norman Gardner Meade. (64) Oct.
The Service Power Plant at the World's Fair.* (41) Oct.; (18) Oct. 1; (15) Oct. 7.
Power Station Equipment for the World's Fair, St. Louis.* (27) Oct. 1.
Electric Motors in Machine Shop Service. (14) Oct. 1.
Speed Regulation in Combined Steam and Hydraulic Power Systems. (14) Oct. 1.
The 5 000-Horse-Power Three-Phase Turbo-Alternator at the Porta Volta Electric Supply Station of Milan.* (19) Oct. 1.
The Yorkshire and Lancashire Electric Power Companies.* (Abstract of Paper read before the International Elec. Congress.) (14) Oct. 8.
A Gas Engine Harbor Electric Plant.* (27) Oct. 8.
The Works of the Ontario Power Company of Niagara Falls.* (14) Serial beginning Oct. 8.
Mercadier's System of Attuned Telegraphy.* (Translated from *Prometheus.*) (19) Oct. 8.
Expériences sur les Courants Continus à Haute Tension.* (33) Sept. 17.
Séparation de l'Huile des Eaux de Condensation par le Courant Électrique.* (33) Sept. 24.
Les Alternateurs Westinghouse.* F. Gramyr. (36) Sept. 25.
Sur l'Établissement Rationnel des Dynamos. C. F. Guilbert. (From *Bulletin trimestriel* de la Société amicale d'ingénieurs sortant de l'école supérieure d'Electricité.) (36) Sept. 25.

Marine.

Description of the Electrical Equipment of an Engine Works and Shipyard, with Notes Thereon. H. O. Wraith, Assoc. M. I. E. E. (77) Aug.
The Austrian Lloyd's Steamer *Africa.** (11) Serial beginning Sept. 2.
The Indian Troopship *Dufferin.* (12) Sept. 16.
Launch of the First Turbine-Propelled Liner for Transatlantic Traffic. (46) Sept. 17.
Steam Turbine Propulsion for Marine Purposes. A. Rateau. (Paper read before the Inst. of Naval Archts.) (19) Serial beginning Sept. 17.
The Design of a Launch Motor.* Edward Butler, M. I. M. E. (47) Serial beginning Sept. 24.
The Design of a Dry Dock.* Alfred W. Young. (Paper read before the Junior Inst. Engrs.) (11) Sept. 30.
Auxiliaries of a War Fleet; with Special Reference to Recent Additions to the British Navy.* Archibald S. Hurd. (10) Oct.
Launching of the *Connecticut* and *Milwaukee.** (46) Oct. 1.
The Launching of the U. S. Battleship *Connecticut.** (20) Oct. 6.

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Marine—(Continued).

Italian First-Class Battleship *Regina Margherita.** (19) Oct. 8.
Some New Types of Lifeboats.* (46) Oct. 8.
Peary's New Ship for Work in Arctic Seas.* (46) Oct. 8.
Ponts Roulants Électriques, pour Cales de Construction de Navires.* (34) Sept.

Mechanical.

Inertia in Gas and Steam Engines.* A. T. Kasley. (58) July.
A Simple Dynamometer.* W. J. Spiro. (6) July.
The Utilization of the By-Products in the Manufacture of Portland Cement. Bernard Enright. (67) Aug.
Flame Temperatures in Internal Combustion Engines. (22) Aug. 26; (26) Sept. 2.
The Utilisation of Waste Heat from Coke Ovens. (22) Aug. 26.
The Aerial Ropeway at Aberfeldy.* (12) Sept. 2.
The Utilisation of Coke Oven Gas.* (12) Sept. 2.
Tandem Compound Drop Valve Steam Engine.* (12) Sept. 2.
The Zoelly Steam Turbine.* Walter Rappaport. (26) Sept. 22.
The Calorimetry of Gaseous Fuels.* W. Garnet Wernham. (Abstract of Paper read before the Junior Inst. of Engrs.) (47) Sept. 8.
Flame Temperatures in Internal-Combustion Motors.* Dugald Clerk. (Paper read before the British Assoc. for the Advancement of Science.) (47) Serial beginning Sept. 8.
The Effects of Receiver Drop in a Compound Engine. Joseph W. Hayward. (Paper read before the British Assoc. for the Advancement of Science.) (47) Sept. 8; (73) Sept. 16.
Machine Tools at St. Louis Exhibition.* (22) Sept. 9.
The Prevention of Heat Radiation from Steam Pipes.* Francis H. Davies. (73) Sept. 9
A New Oil Engine.* (12) Sept. 9.
Notes on American Steel Works.* David E. Roberts, M. Inst. C. E. (11) Serial beginning Sept. 9.
The Turbo-Electric Power System in Paint Manufacture.* (27) Sept. 10.
Effective Steam Plants for Gas Manufacture. C. H. Pennoyer. (Paper read before the Pacific Coast Gas Assoc.) (24) Sept. 12.
Boiler Scale and the Transmission of Heat.* A. D. Risteen. (62) Sept. 15.
A Bricking Plant for Flue Dust and Fine Ores.* Jas. C. Bennett. (16) Sept. 15.
Tests of an Oil Engine with Steam Injection.* (13) Sept. 15.
The 40-Ton Guy Derricks at the New Wanamaker Store, Philadelphia, Pa.* (13) Sept. 15.
Latest Design of American Steel Plant for the Production of Billets.* (22) Sept. 16.
The Clarkson Steam-Motor Omnibus.* (12) Sept. 16.
Trial of a 240-H. P. Diesel Oil Engine.* Michael Longridge, M. Inst. C. E. (47) Sept. 17.
The Crocker Steam Turbine. (47) Sept. 17.
Proportions of the Mechanical Equipment of the Whitehall Building, New York.* (14) Sept. 17.
A Molasses Pumping Plant in Boston.* (14) Sept. 17.
Compressed Air and Pneumatic Tools in Engineering Works. Frank Fielden. (62) Sept. 22.
A Forty-Passenger Automobile.* (20) Sept. 22.
Standard and Systematic Methods for Making Beds.* Thomas D. West. (Paper read before the Amer. Foundrymen's Assoc.) (11) Sept. 23.
The Manufacture of Small Briquettes.* Dr. Steger. (From *Stahl und Eisen.*) (57) Sept. 23.
Valve Motion: Its Relation to Steam Economy.* (Abstract of Paper read before the Traveling Engineers' Assoc.) (40) Sept. 23.
Imperial Works of the Oil Well Supply Co., Oil City, Pa.* (14) Sept. 24.
Suction Gas Producers.* Oskar Nagel. (27) Sept. 24
An Automatic Lamp Lighting and Extinguishing Device.* Alfred Gradenwitz. (19) Sept. 24.
Turbine Testing Plant of the Westinghouse Machine Co.* (14) Sept. 24.
Internal Combustion Engines at the St. Louis Exposition.* (27) Sept. 24.
Some Practical Observations on the Flow of Gases in Pipes. (24) Sept. 26.
Reeves Engine Tests.* (62) Sept. 29.
80-Inch Double Boring and Turning Mill.* (12) Sept. 30.
The Utilisation of Peat.* (22) Sept. 30.
The Economy of Feed Water Heating. (64) Oct.
A Solution of an Important Gas Engine Problem. (64) Oct.
Steam Turbine Construction.* O. Lasche. (64) Oct.
The Riedler-Stumpf Turbine and its Applications.* (64) Oct.
Portland Cement Crushing and Grinding Machinery.* Boilleau and Lyon. (60) Oct.
Fuel Gas for Internal Combustion Engines: Notes upon its Generation and Uses.* J. R. Bibbins. (10) Oct.
Condensing Plant: Types of British Condensers and Accessories.* W. H. Booth. (10) Serial beginning Oct.
Important Improvements in Planer Design. (25) Oct.

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Mechanical—(Continued).

The Fuel Briquetting Industry: the Materials of which Briquets are Made, Binders, Fuel Value, Shapes of Briquets, Processes and Machinery Used.* John Fulton. (45) Oct.
Lathe Centers.* Frank Kleinhans. (41) Oct.
The Practical Working of High-Speed Steels. Thomas Beasley. (41) Oct.
The Giant Establishment of M. M. Schneider & Co., in France.* L. Ramakers. (41) Oct.
Oil Burning Furnaces.* S. Uren. (Paper read before the Nat. R. R. Master Blacksmiths' Assoc.) (18) Oct. 1; Abstract (15) Sept. 30; (20) Oct. 6.
The Construction of an Indicating or Recording Tin Plate Aneroid Barometer.* N. Monroe Hopkins. (19) Oct. 1.
Electric Motors in Machine Shop Service. (14) Oct. 1.
Retort House Practice in a Small Works. George C. Turner. (Paper read before the Michigan Gas Assoc.) (24) Oct. 3.
The Practical Handling of High Speed Steels. (20) Oct. 6.
Steel Castings. (20) Oct. 6.
Proportions of the Mechanical Plant of the North American Building, New York.* (14) Oct. 8.
Temperature as Applied to the Production and Purification of Coal Gas.* V. F. Dewey. (Paper read before the Michigan Gas Assoc.) (24) Oct. 10.
The Formation of Naphthaline in Coal Gas. Alfred H. White and S. Ball. (Paper read before the Michigan Gas Assoc.) (24) Serial beginning Oct. 10.
L'Élévateur-Transporteur Extensible *Condor*, Employé à la Construction de l'Hotel des Postes et Télégraphes de Gand; Systeme Ch. van Driessche. (30) Aug.
La Fabrication des Chaines sans Soudure par le Procédé Belge du Laminage Annulaire.* Eug. François. (30) Aug.
Fonctionnement de la Vapeur Surchauffée dans les Machines à Pistons.* F. Richter. (37) Sept.
La Force Motrice à l'Exposition de Saint Louis: Installation des Générateurs de Vapeur.* L. Piaud. (33) Sept. 3.
Installations Mécaniques et Frigorifiques de l'Hôtel Astor, à New-York.* (33) Sept. 10.
Les Nouvelles Usines Cail, à Denain (Nord): Société Française de Constructions Mécaniques.* D. Dantin. (33) Serial beginning Sept. 24.
La Turbine à Vapeur Zoelly.* (33) Sept. 24.
Élévateurs et Grues Électriques.* C. Domar. (36) Sept. 25.
Les Voitures Peugeot.* F. Doubrey. (36) Sept. 25.

Metallurgical.

Sixth Report to the Alloys Research Committee: On the Heat Treatment of Steel.* William C. Roberts-Austen and William Gowland. (75) No. 1, 1904.
A Crucible Charge for Gold and Silver in Zinc Ores. E. J. Hall and E. Popper. (6) July.
Methods of Pyrometry.* C. W. Waidner. (58) Sept.
Conduit System of Handling Blast Furnace Stock.* (From *Iron Trade Review.*) (47) Sept. 8.
The Tube Mill in the Transvaal. (12) Sept. 9.
The Theory of the Hot Blast. A. D. Elbers. (62) Sept. 15.
Effects of Annealing on Steel Rails. Thomas Andrews, M. Inst. C. E., and Charles Reginald Andrews. (62) Sept. 22.
Pig-Iron in its Commercial Aspects. F. M. Thomas. (Abstract of Lecture before the Birmingham Metallurgical Soc.) (22) Sept. 23.
Specifications for British Foundry Iron. (22) Sept. 23.
Modern Blast Furnace Construction: From the Drawing Office Point of View. J. G. Bayley. (20) Sept. 29.
Ore Mills at the Louisiana Purchase Exposition: The Working Mills and Models Exhibited by California, Utah and South Dakota.* (45) Oct.
The Portland Mill: A Description of the Chlorination Plant of the Portland Gold Mining Co., at Colorado Springs, Colo. F. A. Thomson and S. L. Goodale. (45) Serial beginning Oct.
Influence of Tempering on the Electric Resistance of Steel. H. Le Chatelier. (Paper read before the Académie des Sciences.) (19) Oct. 1.
The Determination of Carbon, with Special Reference to the Stability of Carbon Compounds in Steel.* James A. Aupperle. (20) Oct. 6.
Chlorine in Metallurgy. James Swinburne. (Abstract of Paper read before the International Electrical Congress.) (13) Oct. 6.
La Fabrication du Fer et de l'Acier au Four Électrique.* L. Gautier. (34) Sept.
Recherches sur les Aciers au Titane et à l'Étain.* Léon Guillet. (33) Sept. 17.

Military.

Electrically-Operated Submarine Mines.* Waldon Fawcett. (26) Sept. 2.
Electric Night Sights.* (26) Sept. 9.

Mining.

Safeguards in Bituminous Coal Mining. W. E. Fohl. (58) July.
Underground Wire-Rope Haulage.* (22) Aug. 26.

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Mining—(Continued).

Mine Signalling by Compressed Air.* (22) Sept. 2.
The Applications of Electricity at the Pinxton Collieries.* (73) Sept. 9.
Timber and Timbering.* J. H. Batcheller. (16) Serial beginning Sept. 15.
The South Wilkes-Barre Colliery.* (16) Sept. 22.
Improvements at the Birley Collieries.* (22) Sept. 23.
Pillar Working in Bengal, India. Geo. A. Stonier. (Paper read before the Inst. of Min. Engrs.) (57) Sept. 23.
Oil-Engine, Driven, Pumping and Electric Lighting Plant at Syndale Colliery. Thos. Beach. (Paper read before the National Assoc. of Colliery Managers.) (22) Sept. 30.
Large Blasting Operations in Slate Quarries.* (22) Sept. 30.
The Problem of Dynamic Balance. E. H. Roberton. (Paper read before the Inst. of Min. Engrs.) (57) Sept. 30; (22) Sept. 16.
The Henry Colliery of the Davis Coal & Coke Company: A Description of the Plant and the Mechanical Equipment.* J. F. Healy. (45) Oct.
Suspension des Câbles Électriques dans les Mines.* (34) Sept.

Miscellaneous.

The National Physical Laboratory: Recent Researches and Future Work. R. T. Glazebrook. (Abstract of Paper read before the British Assoc. for the Advancement of Science.) (73) Sept. 2.
The Outlook for Engineering Education. C. Frank Allen, M. Am. Soc. C. E. (Address before the Soc. for the Promotion of Eng. Education.) (40) Sept 16; Abstract (13) Sept. 15.

Municipal.

A Rolling Lift Road.* (67) Sept.
The Development and Importance of Fire Calls and Alarms. James Sheppard, A. I. E. E. (Abstract of Paper read before the International Fire Service Congress.) (26) Sept. 23.
Street Lighting. A. S. Hatch. (Paper read before the International Assoc. of Mun. Electricians.) (60) Oct.

Railroad.

High-Speed Electric Railway Experiments on the Marienfelde-Zossen Line.* Alexander Siemens, M. I. E. E. (77) Aug.; Abstract (25) Oct.
Special Tools in the Elkhart Shops of the L. S. & M. S. Ry.* (39) Sept.
Report of Tank Car Committee, M. C. B. Association.* (39) Sept.; (15) Sept. 23.
Clearance Reducing Valve Mechanism and Cylinder.* (39) Sept.
Dining-Car for the Great Western Railway.* (11) Sept. 2.
New "High-Capacity" Freight Wagons of the Lancashire and Yorkshire Railway.* (22) Sept. 2.
Four-Cylinder Compound Locomotive.* (12) Sept. 9.
The Use of Electricity on the North-Eastern Railway and upon Tyneside.* Charles H. Merz and William McLellan. (Paper read before the British Assoc. for the Advancement of Science.) (12) Sept. 9.
The Exhibit of the "Big Four Route" at the St. Louis Exhibition.* H. W. Hanbury, M. Inst. C. E. (11) Serial beginning Sept. 9.
New Type of Compound Locomotive, Adriatic System, of Italy.* (18) Sept. 10.
Further Use of Steel and Reinforced Concrete Ties.* (18) Sept. 10.
German High-Speed Compound Locomotive at the St. Louis Exhibition.* (13) Sept. 15.
Locomotive Coaling Station for the Central Railroad of New Jersey.* (15) Sept. 16; (40) Sept. 16.
The Westinghouse Automatic Air and Steam Coupler.* (15) Sept. 16; (40) Sept. 16; (25) Oct.
United States Standard Postal Car.* (40) Sept. 16.
Mallet Locomotive, St. Louis Exhibition.* (12) Sept. 16.
Henschel Three-Cylinder Compound Locomotive.* (40) Sept. 16.
The Southern Pacific Flat-Bottom Tieplate. (40) Sept. 16.
Rail Fastenings. A. Morrison. (Paper read before the Roadmasters' and M. of W. Assoc.) (40) Sept. 16; Abstract (15) Oct. 7.
Track Improvements and Maintenance Expenses. E. E. R. Tratman. (Paper read before the Roadmasters' and M. of W. Assoc.) (40) Sept. 16.
Modern Locomotives at the St. Louis Exposition.* (46) Sept. 17.
New Design of Superheater for Locomotives.* (47) Sept. 17.
Effects of Annealing on Steel Rails. Thomas Andrews, M. Inst. C. E., and Charles Reginald Andrews. (62) Sept. 22.
Express Passenger Locomotive for the London and South-Western Railway.* (11) Sept. 23.
The New "High-Capacity" Rolling Stock of the Great Central Railway.* (22) Sept. 23.
New York Terminal Station of the Pennsylvania.* (15) Sept. 23; (46) Oct 8.
New Passenger and Freight Terminal of the Toledo Railway & Terminal Company.* (15) Sept. 23.
Concrete Cross-Ties on the Ulster & Delaware.* (15) Sept. 23; (13) Oct. 6.
The Sedalia Shops of the Missouri Pacific.* (40) Sept. 23.

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Railroad—(Continued).

Boston & Maine Ten-Wheelers.* (18) Sept. 24.
Passenger Car Ventilation System of the Pennsylvania R. R.* Charles B. Dudley. (18) Serial beginning Sept. 24.
The Passenger Car Ventilation System of the Pennsylvania Railroad.* (14) Sept. 24; (13) Sept. 29; (15) Sept. 30.
A New System for the Protection of Trains by Engine-Cab Signals.* Emile Guarini (19) Sept. 24.
Rearrangement of Tracks and Station Facilities at the Union Station, St. Louis, Mo.* (13) Sept. 29.
Motor Cars as Feeders to Railroads. A. Campiglio. (From *Bulletin* of the International Ry. Congress.) (15) Sept. 30.
The Westinghouse Automatic Coupler, Draw-Bar and Buffer for Railway Work.* (22) Sept. 30.
Six-Coupled Locomotive for the Pekin Syndicate Railways.* (11) Sept. 30.
New Locomotive Works in Canada.* (12) Sept. 30.
New Locomotives for the Southern Pacific.* (15) Sept. 30.
The Ganz Three-Phase Electric Railway System.* G. Leve. (15) Sept. 30.
Lehigh Valley Shops at Sayre.* (40) Sept. 30.
Wooden Crossties in Tropical Climates. F. J. E. Spring, M. Am. Soc. C. E. (Abstract of Report to be presented before the International Ry. Congress.) (40) Sept. 30.
The Carter Continuous Track Derail.* (40) Sept. 30.
The Development of the Compound Locomotive in England.* W. E. Dalby. (9) Oct.
Six-Coupled Passenger Locomotive, Boston & Maine Railroad.* (25) Oct.
Steam Motor Cars: Great Central Railway.* (21) Oct.
Electrical Motor Truck: North Eastern Railway.* (21) Oct.
The Adlake Acetylene Car Lighting System. Ed. L. Langworthy. (Abstract of Paper read before the Southern and South Western Ry. Club.) (21) Oct.
2nd Class Carriages: Belgian State Railways.* (21) Oct.
Uniform Code of Signalling by Detonators. (21) Oct.
8 Coupled or "Consolidated" Engines: Japanese Imperial Government Rs.* (21) Oct.
20-Ton 4-Wheeled Hopper Wagon: Great Central Railway.* (21) Oct.
Express Passenger Engines (4-4-0 Outside Cylinders Class): Great Western Railway.* (21) Oct.
Impressions of Foreign Railroad Practice. (25) Serial beginning Oct.
New Locomotive and Car Shops, McKees Rocks, Pa., Pittsburg & Lake Erie Railroad: Blacksmith Shop Equipment.* (25) Oct.
Piston Valves with Relief Plates, Pennsylvania Railroad.* (25) Oct.
The Cross-Compound Locomotive: Distribution of Power.* W. L. Kinsell. (25) Oct.
The New York Improvements of the New York, New Haven & Hartford Railroad. (14) Oct. 1.
Standard Tunnel Plans, New York Central R. R.* (18) Oct. 1.
The Locomotive Exhibits at the St. Louis Exhibition.* (13) Oct. 6.
Controlled Manual Block Signals: Illinois Central.* (40) Oct. 7.
Trolleys and Hoists in Railway Shops.* (40) Oct. 7.
Progress on the Washington Tunnel.* (15) Oct. 7.
Special Horse Car for the Central Railroad of New Jersey.* (15) Oct. 7; (40) Oct. 7; (18) Oct. 8.
Tie Plates. (Report by Com. Roadmasters' and M. of W. Assoc.) (15) Oct. 7.
New Locomotives and Motor Cars for the Paris-Orleans.* (15) Oct. 7.
Improvements on the Philadelphia, Baltimore and Washington.* (15) Oct. 7.
Tank Suburban Engine—Long Island Railroad.* (18) Oct. 8.
Le Chemin de Fer Métropolitan de Paris.* J. Hervieu. (35) Serial beginning Sept.
Rupture des Tubes Indicateurs de Niveau d'Eau, Dispositions Adoptées à la Compagnie du Nord pour Mettre le Personnel des Locomotives à l'Abri des Accidents Qui en Résultent.* M. Defauconpret. (38) Sept.
Le Tramway Électrique du Mont-Blanc: Section du Fayet-Saint-Gervais à l'Aiguille du Goûter.* G. Espitallier. (33) Sept. 3.
Les Grandes Vitesses sur les Chemins de Fer.* R. Bonnin. (36) Sept. 10.
Les Nouvelles Locomotives Compound du Chemin de Fer de Ceinture, à Paris.* F. Barbier. (33) Sept. 10.
Le Funiculaire de la Bourboule (Puy-de-Dome).* A. Dumas. (33) Sept. 17.
Le Saturateur A. Heintz pour le Chauffage par la Vapeur sans Pression.* L. Brossement. (36) Sept. 25.

Railroad, Street.

Present Electric Railway Practice. Clarence Renshaw. (58) July.
Mersey Railway: Multiple Control.* H. L. Kirker. (77) Aug.
The Best Form of Current for Light Railways. M. Pforr. (Abstract of Paper read before the International Union of Tramways and Light Rys.) (26) Sept. 2.
Side-Slip in Motor-Cars.* Horace Darwin and C. V. Burton. (Paper read before the British Assoc. for the Advancement of Science.) (11) Sept. 9.
North Metropolitan Electric Power Supply and Tramways.* (26) Serial beginning Sept. 9
A New Sub-Station on the Central London Railway.* (73) Sept. 9.
A New American Single-Phase Motor.* (73) Sept. 9.

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Railroad, Street—(Continued).

Single-Phase Railway Motors of the Oerlikon Company.* (73) Sept. 9.
An American Single-Phase Railway.* (47) Sept. 10.
History and Physical Aspects of the New York Subway.* (15) Sept. 16.
Traffic Features of the Subway.* (15) Sept. 16.
Signaling in the Subway. (15) Sept. 16.
The Power Equipment of the New York Subway. (15) Sept. 16.
Tests of the Insulation Resistance of Electric Tramway Systems and Maintenance of the Trolley Wire. (17) Sept. 17.
The Dayton, Covington & Piqua Traction Company.* (17) Sept. 17.
A Trolley for Heavy Multiple-Unit Electric Trains.* Herbert I. Bennett. (46) Sept. 17.
Notes on the Work of the Electric Railway Test Commission.* Will Spalding. (72) Sept. 20.
Hydraulics in Connection with Electric Railway Work. B. E. Morrow. (72) Sept. 20.
Third-Rail System of the Scioto Valley Traction Co., Columbus, Ohio.* (72) Sept. 20.
The Fayet-Chamonix-Martigni Electric Ry.* Enrico Bignami. (72) Sept. 20.
Power Facilities and Car House Practice Followed by the Public Service Corporation (of New Jersey.) (72) Sept. 20.
Mill Construction Car House and Office at Bristol.* (72) Sept. 20.
Notes on the Power Stations of the St. Louis Transit Co.* Richard M'Culloch. (72) Sept. 20.
McKinley Syndicate Properties of Central Illinois.* (72) Serial beginning Sept. 20.
Maintenance of Electric Cars and Their Equipment. H. A. Benedict. (Abstract of Paper read before the St. Ry. Assoc. of the State of N. Y.) (72) Sept. 20.
Block Signaling of Electric Railways with Track Circuit Control. J. B. Struble. (Paper read before the St. Ry. Assoc. of the State of N. Y.) (72) Sept. 20 (17) Sept. 24.
Vestibule Cars for Chicago Metropolitan Elevated.* (72) Sept. 20.
Hadfield's Special Steel Track-Work.* (11) Sept. 23.
South London Conduit Tram Lines.* (12) Sept. 23.
New Terminal Facilities at the Minnesota State Fair Grounds.* (17) Sept. 24.
Power Production and Distribution on the Metropolitan Street Railway System, New York. M. G. Starrett. (Paper read before the N. Y. State St. Ry. Assoc.) (14) Sept. 24; (17) Sept. 21.
Cars: Their Equipment and Maintenance (on the Nottingham System). John Aldworth. (Paper read before the Municipal Tramways Assoc. of Great Britain.) (73) Sept. 30; Abstract (26) Sept. 30.
Steel Cars for the New York Subway.* (15) Sept. 30; (25) Oct.
High-Speed Electric Interurban Systems for Freight and Passengers.* Ralph Scott. (9) Oct.
The Power House of the Subway Division, Interborough Rapid Transit Co., New York.* (14) Serial beginning Oct. 1.
Passenger Stations and Engineering Details of the New York Subway System.* (17) Oct. 1.
General Design and Finish of the Modern Interurban Car.* Edward C. Boynton. (17) Oct. 1.
New Form of Sprague-General Electric Automatic Relay Train Control for Boston Elevated Railway.* (17) Oct. 1.
Power Used by Brakes. (17) Oct. 1.
Signals in the Subway.* (15) Oct. 7.
Some Early Electric Railway Generators.* A. L. Rohrer. (27) Oct. 8.
Underground Rapid Transit in New York City.* (27) Serial beginning Oct. 8.
Viaduct Erection on Section 10, New York Rapid Transit Railroad.* (14) Oct. 8.

Sanitary.

Centrifugal Fans and Their Application to the Heating of Industrial Buildings. R. B. Hayward. (58) June.
French Heating Methods.* G. Debesson. (70) Sept.
The Ventilation of Retort Houses. Alfred Lotz. (70) Serial beginning Sept.
The Riker's Island Refuse Conveyor Plant.* (14) Sept. 17.
Refuse-Consuming and Gas-Producing Furnace.* (47) Sept. 24.
Reinforced Concrete Sewers in Wilmington, Del.* (60) Oct.
Sewage System for Mining Town.* F. A. Coleman. (45) Oct.
Sewage Purification Works under Construction at Minworth Greaves, England. (14) Oct. 8.
Experiments in Purifying Brewery Refuse. (14) Oct. 8.
Cleaning the Ponds in a Boston Park. (14) Oct. 8.
The Ventilation of the Hotel Astor, New York.* (14) Oct. 8.
L'Humidification des Locaux de l'Industrie Textile.* Paul Razous. (33) Sept. 17.
Chauffage à Eau Chaude, à Circulation Périodique ou Continue, Système Rouquand.* A. Monteilhet. (33) Sept. 24.

Structural.

Preliminary Report of the Committee on the Uniform Analysis of Cement and Cement Materials. W. B. Newberry, Richard K. Meade, and Ernest B. McCready. (Paper read before the Amer. Chem. Soc.) (67) Aug.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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Vol. XXX. OCTOBER, 1904. No. 8.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

SOME NOTES ON THE CREEPING OF RAILS.

Discussion.*

BY MESSRS. G. LINDENTHAL, W. M. CAMP, F. S. STEVENS, HUNTER McDONALD, P. H. DUDLEY AND GEORGE TATNALL.

Mr. Lindenthal. GUSTAV LINDENTHAL, M. AM. SOC. C. E. (by letter).—This subject has been of much interest to the writer for many years, particularly in connection with track over bridges, and has led him to an investigation of the creeping of rails, generally, and the formulation of a theory, which he published,† and from which he desires to quote as follows:

"As is well known, rails creep in the direction of the traffic, whether up grade or down grade. On single track, rails creep in the direction of heavier traffic; otherwise they creep down grade.

"The force with which a track is creeping seems almost irresistible. Notching the rail base for the spikes was one of the means employed to stop it. But the spikes were gradually sawed into, pulled out or sheared off by the creeping rails. Later, when the practice of notching was given up, because steel rails were apt to break at the notched places, spiking of the angle bars at the rail joints was resorted to. Special angle brackets or straps bolted to the rail between joints are also used. But all in vain.

"Several explanations have been given for the phenomenon, but none accounts for all its peculiarities satisfactorily. My own view is that creeping is caused by the push of the rolling friction of the wheels

* This discussion (of the paper by Samuel Tobias Wagner, M. Am. Soc. C. E., printed in *Proceedings* for May, 1904), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Communications on this subject received prior to November 25th, 1904, will be published subsequently.

† *Railroad Gazette*, September 29th, 1899.

Mr. Lindenthal.

on the rails. If, for illustration, we imagine the cars mounted on skids instead of wheels, the creeping would be greater in the proportion as sliding friction is greater than rolling friction. The drivers of the locomotives are the only wheels exerting a pull on the rails, which for the time being are pinned down and so cannot move. The pull is transmitted by friction through the ties to the ballast and earth underneath, the whole forming an anchorage—but a traveling anchorage—always located under the locomotive, whether pulling by adhesion, or through cog-wheels on a rack rail.

"All other wheels behind the locomotive exert a push on the rails through rolling friction in the direction of the train.

"The coefficient of rolling friction varies with the velocity of the train from nearly one per cent. for slow velocities to four per mill for trains at and over 30 miles per hour. It also varies on curves. The average value for fast trains on tangents and curves may be assumed for illustration at 5 per mill. A car wheel loaded with 10 000 lbs. will thus exert a push of 50 lbs. on the rail.

"As the friction between the loaded rail and the ties—disregarding the holding power of the any way imperfectly fitting and usually loose spikes—is at least 30 times greater than the rolling friction of the wheels, the rail could not possibly move or creep under a smoothly rolling load.

"But the wheels do not roll smoothly and moreover they do not roll over continuous rails. The rails are in lengths from 30 to 60 ft., with space between the ends for expansion. The wheels get upon each rail length suddenly, and instead of a quiet push or a static force of 50 lbs. per wheel, there is a horizontal blow or a dynamic force, the effect of which increases as the square of the velocity of the train.

"It should be understood that this horizontal blow is not caused by the depression of the rail ends or by the space left for expansion between them, although both these features, when they exist, undoubtedly augment its force. It is unavoidable with the smoothest and any kind of rail joint (including the joint in which the outside splice bar carries the wheels over the rail ends) because the longitudinal strain is set up suddenly in each rail length, and cannot be transmitted to the next rail, unless the rail ends were tightly pressed together, equivalent to a continuous rail.

"Taking 40 miles per hour, a usual speed for fast freight trains, equal to 60 ft. per second, the impact from the rolling friction of the wheel will be $50 \times 60^2 = 180\,000$ foot-pounds, which in effect is the same as if an iron ram, weighing one ton, would hit the end of the rail with a velocity of 9½ ft. per second. The car wheels being spaced 5 to 20 ft. apart, will deliver upon the receiving end of a 30-ft. rail from 3 to 6 such blows per second. If the rail is not rigid and smooth, if, on the contrary, by reason of defective or yielding supports, it deflects and by its resilience causes the wheels to rebound and ricochet, so that they no longer roll with uniform pressure, then each wheel will hit the rail as many additional horizontal blows as it makes jumps or rebounds. A rebound every few feet is an ordinary occurrence on spiked track. A 30-ft. rail may thus receive from 30 to 60 glancing blows per second, and from a train of 40 cars a total of 1 500 to 3 000 blows, varying in effect from a few foot-pounds to 180 000 and more per blow, lengthwise with the rail and in the direction of the train.

"With this explanation it is not difficult to understand why the creeping should seem irresistible, the rails shearing off spikes and bolts and wandering up steep grades. In principle, to make it plain, Mr. Lindenthal. it is the same dynamic effect with which the blows from a 10-lb. hammer will suffice to overcome the friction (of over 5 000 lbs.) of a rail spike in an oak tie.

"A rail may be held down by spikes and by the moving loaded cars, but it will nevertheless be displaced by their dynamic effect in the manner stated.

"Each rail moves individually and independent of the adjacent rails. The play left for expansion at the joints facilitates the creeping. There could be no creeping with a continuous rail.

"All observed facts can now be satisfactorily explained. There is no creeping in switch yards or on side tracks, on which trains move slowly and in opposite directions. Compared on the basis of ton-mileage, the rate of creeping is greater on tracks for fast trains, and with short or light flexible rails, and on poorly ballasted and yielding foundations. Every cause which induces or increases rebounding of the wheels (as poor rail joints) will increase creeping.

"On down grades, where the locomotive is not pulling, the effect of its wheels will be added to that of the car wheels, and on that account and by reason of the greater velocities on down grades, the creeping will be greater. So also the outside rails on curves creep faster than the inside rails, because exposed to greater pressure and friction, causing harder glancing blows."

Those who are interested in the subject may follow the writer's article further on the remedy for creeping, etc.

The only correction that the writer would make in the foregoing citation is in the coefficient of rolling friction. Five per mill is probably true enough for rough defective track; on first-class track, with 100-lb. rails, the rolling friction between wheel and rail may be only one-tenth as large. Data on rolling friction are still unreliable and divergent; but with poor rail joints, leaving the receiving rail and sticking up like a step, it will receive something more than a glancing blow. The assumption of 5 per mill rolling friction, in such case, will not require reduction.

The explanation of creeping, cited by Mr. Wagner from Camp's "Notes on Track," that the principal cause is "the wave motion in the rails set up by moving trains," is, in view of the foregoing, as little satisfactory as would be the explanation for the progress of the locomotive being caused by crank motion. It stops too far away from the primary cause, which is of a dynamic nature. If the rails consisted of discontinuous high stiff girders on solid continuous bearings, as, for instance, on longitudinal concrete beds, which construction could not be subject to wave motion, there would still be creeping of rails, although at a diminished rate. As long as rails may move on their bearings from temperature changes, so long they may move from the rolling friction of the wheels, which is the explanation for creeping, as stated above.

Mr. Camp. W. M. Camp, M. Am. Soc. C. E. (by letter).—This paper takes up an old, though timely, subject. The misadjustments in track caused by creeping rails cannot be considered new experience. They have been a source of trouble to trackmen since the early days of railroading, to a greater or less extent; and, while the causes, perhaps, may be more widely understood at the present time than was the case a generation ago, there still remain some things to learn concerning the phenomena of rail movements under traffic, in order to seek intelligently the most effective means or methods of prevention. As the creeping action is always more pronounced on double, than on single, track, other conditions being the same, the subject gains importance with time, owing to increase in the mileage of second track; and increase in the weight of rolling stock is another cause which aggravates the trouble. Mr. Camp.

Although, in years past, there has been considerable discussion of rail creeping and the conditions responsible for it, yet the subject was seldom or never investigated with that thoroughness which is characteristic of engineers when setting out to overcome difficulties. Therefore, it is a matter of no little satisfaction that Mr. Wagner has collected exact measurements and other data of rail creeping, in connection with careful observations of the attending conditions. It is only through the publication of such results, and discussion of the whole question from varied lessons of experience, that the problem can be thoroughly comprehended.

The records found in this paper, while not exhibiting extreme cases of rail movement, nevertheless present a variety of conditions and afford good opportunity for studying the subject. The greatest movement seems to have been at Bethayres, on the New York Branch, and at "Head of Grade," on the Frackville Branch. At the former point the movement was $3\frac{1}{4}$ in. in the south rail of the east-bound track, in the direction of the traffic, in a period of 4 months and 9 days; in the north rail of the same track the movement was $2\frac{3}{8}$ in. during the same time.

The difference in movement between the two rails of the same track is only a matter of common occurrence. The larger movement of the south rail may have been caused by its location nearer the outside of the roadbed, where there would be more spring in the bank, and consequently a greater amplitude of wave motion. In some cases, also, the spikes on one rail of a track may happen to be staggered in the manner which secures the most effective cross-binding action on the rail, with a tendency to hold it against creeping. This occurs where the outside spike leads in the direction of the creeping.

The roadbed at this point is an earth fill over a marsh, a combination of two conditions quite conducive to rail creeping. The rails in the two tracks were of the same weight, and the traffic in the two directions was practically the same, in regard to the number of both passenger and freight trains, but the rails in the west-bound track were held by slot-spiking at the splice-bars, whereas such was not the

Mr. Camp. case in the east-bound track. This fact will account, in some degree, for the smaller movement of the rails in the west-bound track, where the creeping amounted to only from $\frac{5}{8}$ to $\frac{7}{8}$ in. during the period of the observations.

Another cause for the difference might have been a variation of speed in the trains in the two directions. The point of observation was in a sag, at the foot of a grade of 0.7 %, about 3 miles long, meeting a level stretch of track, 3 000 ft. long, across the marsh. It is reasonable to suppose that at least the freight trains on the east-bound track, approaching the point of observation on a descending grade, would pass the point at higher speed than trains of the same class on the west-bound track. The comparison of the creeping action on the two tracks, therefore, is corroborative of explanations which are well understood.

The case next in importance, measured by the extent of the creeping action, seems to have been at "Head of Grade," on the Frackville Branch, where the west rail of the south-bound track crept $3\frac{5}{8}$ in. during the time between February 16th and September 28th, but, during the same time, the east rail of the same track crept only $1\frac{1}{4}$ in. The creeping on the north-bound track at the same point was unimportant. The grade at this point is 3.3 % for a distance of 3.16 miles. Examination of Table 4 shows that the joints on both rails of the north-bound track were slot-spiked, whereas on the south-bound track only occasional joints were held in this way. The traffic, both passenger and freight—but with the freight trains greatly predominating—seems to have been the same on both tracks. It might be expected, however, that the creeping force would be much greater on the south-bound track, on which the trains run with the grade, and on which faster speeds are undoubtedly made than on the north-bound track, where the train movements are against the grade. It is also stated that the freight trains on the north-bound track consisted almost entirely of empty cars. Here, again, therefore, the observations and accepted theories applying to the conditions are in harmony.

At Broad Mountain the conditions respecting grades and traffic seem to be substantially the same as at "Head of Grade," with similar, but not as serious, results.

It appears from the diagrams that in several places the movement of the rails was opposite to the direction of the traffic. In every case of this kind, however, the extent of the movement was small, being most at Hopewell, where it was $\frac{3}{4}$ in. in the north rail of the east-bound track and $\frac{1}{2}$ in. in the north rail of the west-bound track. At North Wales both rails of the north-bound track, apparently, moved in opposition to the traffic, the maximum contrary movement there being about $\frac{3}{16}$ in.

At a number of other points there was apparently a slight movement in the direction opposite to that of the traffic, but in every case the extent of the movement was so small, and the movement of the rails in either direction, at those points, so small, that it may reasonably be doubted whether rail creeping, proper, took place at all. The writer is inclined to think that the movement in such cases was only the effect of local expansion or contraction, from change of temperature, which might have been stronger than the tendency of the rails to creep under the traffic. Mr. Camp.

Rail creeping causes trouble and expense in various ways. The movement shoves joint ties off their tamped beds upon loose ballast, which affords inferior support; the slewed ties must be respaced, to put them square with the track or to restore the proper space intervals; frogs are sometimes crowded out of alignment; and the creeping action closes joint openings necessary for the free expansion of the rails with rise of temperature. Other well-known evils might be mentioned, but the foregoing will suffice to indicate the relative importance of this matter in track work.

In seeking to arrest rail creeping, it is expedient, for practical considerations, to be able to connect the effect with the cause, for it is well understood that the tendency for rails to creep is greater in some localities than in others. Observers have come to know that the character of the ground formation or roadbed is one of the most important conditions accountable for the creeping movement. In using means of prevention, therefore, it is essential that knowledge of conditions of this kind, in some detail, should enter into the considerations. In practice, trackmen have found it possible to stop the creeping at some points, while at other points the same methods were entirely inadequate to overcome the trouble or even alleviate the situation in any material respect. Hence the economical aspects of the question also call for knowledge of the local conditions. It follows from this that a concentration of appliances at those points where the tendency to creeping is strongest may be more effectual than an application of any particular device uniformly at all points where creeping occurs. That is to say, the creeping of rails at all points along a stretch of track of assumed length might be due to conditions which exist along only a portion of the distance. The creeping action at the particular points or localities where the propelling force is engendered may be strong enough, if not opposed, to shove the line of rails over the whole distance considered. The records and diagrams presented by Mr. Wagner show that the rail creeping at some points was negligible, while at other points it was not entirely preventable by the means applied to overcome it, although they might have retarded it to some extent.

In his paper, Mr. Wagner has been kind enough to refer to certain characteristic facts of rail creeping put into formulated expression by

Mr. Camp. the writer after some years of reflection on experience. It will not be necessary, therefore, to repeat the writer's opinions on the general principles of the question except where special comment is intended. One of the principles laid down is that rails creep more on newly-made embankments than on old roadbed which has become solidly compacted. By way of illustration, a case will be cited which came within the writer's experience at one time on the Pennsylvania and New York Division of the Lehigh Valley Railroad. The original road-bed on this line was the tow-path of the abandoned Pennsylvania and New York Canal, which had become well settled under the tramping of boating teams, and from weather exposure for many years. Eventually, the road was double-tracked, the second track at almost all points being laid upon a new embankment made by filling upon the bed of the abandoned canal. At one location, in particular, covering a stretch of track some 4 miles in length, this new embankment was continuous, and upon it was laid the new north-bound track, the old track then being used for the south-bound traffic. The grades were light enough to be fairly negligible in the present connection, and the number of train movements was about the same in each direction, the weight of traffic being somewhat heavier north bound, owing to the movement of loaded coal trains in that direction, although the speed of these loaded north-bound trains was slower than the empty south-bound coal trains. The height of the new embankment was about 8 ft. and upward. On both tracks the joint splices were slot-spiked to the ties, and both tracks were ballasted with gravel of good quality. The exact weight of the rails on the two tracks cannot now be recalled, but those on the north-bound track were new and of heavier section than the old rails of the south-bound track, the weight of which was somewhere between 50 and 60 lb. per yd.

As soon as the road began to be operated as double track, excessive creeping was observed on the north-bound track, and all measures taken to stop it were ineffective. In six months, during the summer, the rail movement, by actual measurement, was 22 in. The creeping action was so strong that both ties at all the (suspended) joints were carried bodily with the rails, so that respacing of these ties became necessary long before the whole movement noted had taken place. On the old roadbed (south-bound track) the creeping movement during the same time was only 3¼ in. During the following year the creeping on the north-bound track was not nearly as much, and the comparison of the rail movements of the two tracks was not nearly as marked, which was undoubtedly due to the fact that, as the new road-bed became consolidated, the force causing the creeping gradually decreased.

In regard to grades, the writer is of the opinion that they are a factor in rail creeping only as far as they influence the speed of trains. On single track he has observed rail creeping both up and down grade, and, in one case, which is worth citing, the rails crept in both directions on the same grade. This was on the Seattle, Lake Shore and Eastern Railroad (now the Seattle and International Railway, a branch of the Northern Pacific Railway), on a grade about 5 miles long. Over nearly the whole length of this grade, from the top, the rails crept down hill, presumably because of the faster speeds in that direction. For a distance of about ½ mile from the foot of the grade, however, the rails crept up hill. The explanation was found in the high speed of loaded logging trains "running for the hill" when beginning to ascend the grade. At the foot of the grade the creeping movement up hill amounted to 2 in., in one season; but, farther up the grade, the creeping action gradually grew less and less, until, as already stated, it died out altogether in a distance of about ½ mile. In this case it seemed clear that, as the speed of the heavily loaded trains slackened in working against the grade, the rail creeping up grade gradually decreased in correspondence, until a point was reached where it was counteracted by the opposite creeping action caused by the faster speeds on the down grade. Mr. Camp.

F. S. STEVENS, M. AM. SOC. C. E. (by letter).—As the writer understands the matter, the term, "creeping," only applies to those cases where the rails are moved by external forces; and movement which is due to expansion and contraction should not be considered. The only force, then, that can produce the trouble is the moving train, and, as the friction or adhesion will hold the rails in place on the ties, it follows that this movement must occur where the rail is not loaded and is practically free from contact with the ties, and, therefore, it is due to the wave motion caused by the weight of the train passing over a roadbed which yields more or less to the compression and is somewhat elastic. The movement, then, must precede the train, and will increase directly with the elasticity of the roadbed and the condition of the track as to surface. The writer has had charge of track over ground which, apparently at some time, had been swamp, and the soft alluvial formation, of great depth, carried the embankment of gravel, not exceeding 6 to 8 ft. high; but this comparatively thin layer of gravel was subjected to great vibration under passing trains, and the wave in front of the engine was very perceptible. The result was that frequently the splice-plates were broken at the center, and the rails separated from 6 to 12 in., the splice-plates evidently having cracked some time before, and the joint parted in this way; and if there were no cracked plates to part and thus relieve the strain, the bolts were sheared off and the same opening appeared. The strange fact, in connection with the parting of these splices, was that the joints in the track on the hard ground adjoining did not close, but remained open for a distance of ½ mile or more from the old swamp, and there the rails ran close together and showed a tendency to Mr. Stevens.

Mr. Stevens. buckle. The movement was in the direction of the heavier traffic and up grade.

Similar conditions have been found where new ties have been "spotted" in at intervals and have not received proper attention within reasonable time afterward. These, becoming loose, hung by the spikes and gave the rails so little support that violent wave motion was set up and the consequent creeping took place.

The effect of loose ties is more noticeable where the tracks are curved, for where the traffic is heavy and slow, and trains are long, and where the track is surfaced for high speed, the load shifts to the low rail and the wheels that travel on the high rail must slip; and, in addition to the wave motion, there is the dragging effect of the sliding wheels, with the result that the outer rail of the curves creeps and the inner rail does not, to any great extent.

Loose spice-plates are also a factor in the creeping of rails, and, where other conditions are good, loose splices alone will cause some movement. The remedy is to prevent, as far as possible, all wave movement by using rails of sufficient weight to carry the loads without appreciable deflection, build firm and unyielding roadbeds, and use enough good ballast to distribute the load properly and see that it is kept open, so that all water may escape quickly and without saturating and softening the roadbed; then keep all ties tamped to a true and uniform bearing, each tie carrying its proper proportion of the load. The rails in such track will creep very little, and no trouble will be found in keeping the proper expansion allowance at the joints, if the splices are kept uniformly tight.

Mention may be made of the Chicago and Alton Railroad, on which rails 133 ft. in length are being used, and it is understood that there is no trouble with them. Really, the only question, in this matter of length of rail, is that of handling, and that should not cause much trouble to roads close to their source of supply. The writer does not believe that with tracks there has ever yet been any trouble which could be traced to excessive length of rail, but there has been a great deal of trouble caused by creeping rails which crept because they were neglected, or the ties were loose, the joints were loose, the surface poor, or the men in charge incompetent. When engines and cars ride smoothly, without jolt, jar, sway, or lurch, the rails are not creeping to any great extent. It is poor track which does the creeping, unless the trouble lies below the bottom of the roadbed. Drainage is the "whole thing" in making and maintaining good track which will hold its surface and stay where it is placed, and the cost of maintenance varies, as to rail and labor, very nearly in proportion to the tonnage moved. Eliminate the wave motion by thorough drainage, sufficient weight of rail, good ties, and good surface, with tight joints, and no fear need be entertained on account of trouble caused by creeping rails.

HUNTER McDONALD, M. AM. SOC. C. E. (by letter).—The writer's experience with creeping rails has been confined almost entirely to single track, while the observations described by the author were made exclusively on double track. Mr. McDonald.

The creeping of rails on single track presents also many apparently inexplicable phenomena. It has been the writer's experience that, no matter what the direction of the preponderance of traffic, the rail will often creep down grade in both directions until all the joints in the sag are closed up. Gravity, no doubt, has much to do with this, but the writer believes that the temperature has also a large share in it. The rail is first laid with proper provision for expansion. As expansion takes place the rail moves more easily down the grade. When contraction follows, the rail is again drawn down the grade rather than up.

The creeping of rails against the current of traffic, as shown in many instances on the author's cards, often occurs for a short period, but it seldom continues unless some condition of loose track, easily discernible, brings it about. This creeping is probably due to temperature changes.

The writer recalls many instances of creeping rails, one of which may be worth mentioning. It was on a trestle about 4 000 ft. long. Near its middle the track was a level tangent. The south half was on a 1.5% grade, ascending from near the middle, and included a long 6° curve to the east. The north half was on a 0.6% grade, ascending from near the middle, and included two short, sharp curves near the north end, reversing on a point. The excess of traffic was southward. A Wharton switch turned out near the middle on the west side, the frog being rigidly fastened to the stringers. In earlier days the trestle had few braces in the direction of the track, and vibrated greatly under trains. The east rail crept northward for years at an average rate of $\frac{3}{4}$ in. per day. Efforts to stop it by fastening the rail always resulted in shearing the bolts or moving the ties, no matter how well fastened down. Until the method of using pointed rails was adopted, it was the practice to put in long rails at the south end and short ones at the north end every Saturday afternoon. The west rail never gave any trouble whatever from creeping in either direction. It was on the outside of the long 6° curve. All creeping was effectually stopped when heavier stringers were put in and every eighth bent braced securely by two heavy pieces of timber fastened on each side of the cap and running to the sills of the second bents on each side.

It has been the writer's experience that the rail always creeps on, and in the vicinity of, long trestles, especially those providing for large waterway across bottoms, and in which, on account of driftwood, it is not practicable to maintain much longitudinal bracing.

When some method of fastening the rail securely to the tie is de-

Mr. McDonald. vised, more rigid track can be maintained, and trouble from creeping greatly reduced.

The anti-creeping devices, designed upon the wedge principle, will hardly take care of cases where the rail creeps in both directions, and will not be needed, to any great extent, when the present method of spiking gives way to one that will hold the rail tight down to the tie.

Mr. Dudley. P. H. Dudley, Esq. (by letter).—The measurements of the creeping of rails, by Mr. Wagner, are important, because they cover so many locations and conditions of service. These will lead to others of as high a degree of excellence, and the principal conditions incident to creeping rails, some of which are local, will become evident.

When the writer's "questions" for the International Railway Congress were prepared and sent to the railroads, it was expected that a number of similar observations would be secured from several railroad companies. The replies, without measurements, were so discordant, owing to different conditions, that they could not be used to determine the local causes as to the creeping of the rails. After the questions had been sent out, the Permanent Committee of the International Railway Congress restricted the reports to a less number of pages than expected; therefore, detailed observations could not be reported, on subsidiary subjects. It is well that these are presented where they will receive adequate publication and discussion.

In Mr. Wagner's records of the Atlantic City Railroad, the creeping of the rails was not decided, for trains exceeding one mile per minute, from January 14th to May 7th, at Clemmenton, Albion, Farmington, and in part at Pleasantville, the movements being due principally to the elongation and contraction from thermal stresses. From May 7th to November 24th, the movements were larger. The details, as to gradients and character of the track, are given for all the different points of observation. At Clemmenton, on the south track, to May 7th, the west or right rail had moved in the direction of the traffic 0.26 in. From May 27th to November 24th, a part of this was lost, the rail moving in the opposite direction, the final movement being 0.20 in., compared with the observation of January 14th. The east or left rail did not show much movement until after May 17th, when the total movement to the left was 0.28 in.

On the north-bound track, to May 7th, the movement of the left rail seems to have been due principally to the adjustment of thermal stresses, and by May 27th it had moved against the traffic 0.16 in., but by November 24th it had moved 0.86 in. with the traffic.

The right or east rail, on May 27th, had moved 0.08 in. against the traffic, though by November 24th it had moved 0.20 in. with the traffic. This, under high-speed trains, is a limited amount.

At Albion the movement of the rails on the south-bound track was down grade against the traffic. On the north-bound track it was down grade also, though the total movement was slight. From May 7th to November 24th, for the left rail it was 0.36 in. and for the right rail 0.42 in. Mr. Dudley.

At Farmington (level track) the right rail on November 10th had moved 0.1 in. with the traffic, the left rail 0.6 in. On the south-bound track, both rails, from May 8th to November 10th, moved only 0.22 in.

Examination of the figures giving the movement of the joints on either side of the observed lines shows, by the rails rendering in the splice-bars, that more or less motion was taking place. The friction of the 90-lb. splice-bars against rendering, in the testing machine at Watertown, was about 4 500 lb. per lin. in., or 63 000 lb. for 14 in. in one rail end, when the bolts were tightened up as by the trackmen.

At Pleasantville, on the north-bound track, from January 15th to November 10th, the right rail moved 0.36 in. with the traffic, and the left rail 1.14 in. On the south-bound track the left rail moved 1.80 in. with the traffic, the right rail moved only 0.56 in. At these two locations the rails are 45 ft. in length.

It is usually found that the 45 and 60-ft. rails are slightly looser under the spikes than the shorter rails.

The excess tendency of the right or left rail to move under the fast trains is indicated there by the greater movement for the left rail. This applies only to similar conditions, for the observations at other points show the greater creeping by the right rail.

The movement of the cross-ties is not noted, but was slight, if any, except for those to which the angle plates at the joints were spiked, the rails running under the spikes. Where the ballast is out, the cross-ties move, or permit more rapid running of the rails.

In the winter the tendency of the thermal stress is to cause tension in the metal of the rails, while in the summer it is compression, before or while the rail ends are rendering in the splice-bars, for adjustment. In the first condition the rails are not as loose as a continuous member under the passing locomotives and cars, and the unit fiber strains as a rule are not found to be as large, when measured by an instrument, as in the summer. This statement must not be confounded with the fact that under great falling temperatures the combined thermal stress of tension and that due to the positive bending moments in the base of the rail of the passing wheel effects may be and are often greater than those experienced in the summer, under the same locomotives. The thermal strains of tension reduce the factor of safety of the metal, while those of compression increase the looseness of the rails and the disturbance of the line. The tendency of the rails in summer, therefore, is to become slightly looser as continuous members, for the same standards of maintenance, and to creep most under the spikes.

Mr. Dudley. The unit fiber strains under locomotives of two pairs of driving wheels are increased from 20 to 35% by the expended tractive effort, besides that due to the axle loads.

The superstructure of all steam roads is flexible, and is depressed temporarily from the "trackmen's surface" to the lower running surface in the "general depression" under the wheel base of the locomotives and cars. In the "general depression" there are specific deflections, in the rails under the wheels, of positive bending moments, constrained by negative bending moments in the wheel spacing. This constitutes the so-called "wave motion" in the superstructure. The ballast and subgrade are also loaded and partially unloaded, each wheel being distinctly felt in the subgrade, for 15 or 20 ft. in depth, in most places. Each type of locomotive in passing causes a characteristic "general depression" and bending moments, in loading the subgrade, and in extreme cases the rails and cross-ties show a slight movement with each train, and require attention to prevent creeping. Conditions in the permanent way which permit a depression of the rails of ¼ in. or more under the passing locomotives or cars, cause a tendency to creep. The outer rail on curves also tends to creep, with the traffic.

Decreasing the positive and increasing the negative bending moments in the rails, either by increased stiffness, better drained subgrade, a more favorable construction of the locomotive for the distribution of the total load and loading the rail, high standards of maintenance, with ample ballast, increase the combined stability between the passing locomotive and the permanent way, and check creeping. This is proven, in the practice of the past decade, by the fast and heavy trains in daily service, compared with those on the former light rails.

The 4-wheel "leading and guiding truck" for the engine, invented by the late John Bloomfield Jervis, Hon. M. Am. Soc. C. E., one of America's eminent civil engineers, has been the great factor in loading, not only the rails in the "general depression," but the subgrade.

Stephenson's unexcelled work was the combination for rapid steam generation for locomotives, while that of Jervis, his contemporary, the distribution of the load and ease of motion. The mechanical applications of both principles have exceeded the expectations of either, with less creeping of the rails than they experienced.

Mr. Tatnall. George Tatnall, M. Am. Soc. C. E. (by letter).—The author presents a very interesting series of observations on a troublesome subject, on which there are very few accurate data, and yet a great deal of speculation. Almost every maintenance-of-way engineer has his own pet theory of the cause of the creeping of rails, evolved from some personally observed and half-remembered facts.

The minute description of the methods of observation speaks well for the accuracy of the results obtained; nevertheless, there are some matters of importance decidedly lacking. The author quotes from Mr. Camp: "It is now generally conceded that the principal cause of creeping is the wave motion in the rails set up by moving trains." This coincides with the opinion expressed by himself, and yet no effort seems to have been made to ascertain the amplitude of this wave motion, in conjunction with the other observations. Mr. Tatnall.

The worst case of creeping that ever came under the writer's observation occurred on a wooden Howe truss bridge. This structure consisted of thirteen spans, each of about 150 ft., and a draw-span of 80 ft. clear, with 14 ft. between the trusses. The floor system was constructed of cross-beams of 8 by 16-in. yellow pine resting on the lower chord and spaced 16 in. apart, on which rested a longitudinal stringer of 8 by 12-in. yellow pine laid flat, on which the rail was spiked. This bridge carried a very heavy traffic, both in the number and weight of trains, and was considerably over-strained, so that constant attention was imperative and repeated repairs and renewals necessary. It has since been entirely replaced with a steel structure.

Upon this bridge, directly under the writer's eye, a creeping of rails of more than 1 in. with the passing of each train has been observed repeatedly. As the bridge was a single-track structure and the trains alternated in direction, the creeping caused by one train was balanced by the creeping in the other direction caused by the next. But this creeping did not occur under the driving-wheels, nor under the coach wheels, which has long been a bone of contention, but occurred entirely at the moment of the passage of the advance upward wave, explained and described so clearly by Captain J. E. Howard, of the U. S. Arsenal at Watertown, Mass., in his reports on measurements of track deflections, and which occurs several feet in front of the engine. At the time of the observation of this excessive creeping, the trusses of several adjoining spans were three or four years old, and the deflection was beginning to be excessive; the deflection of the floor grades had increased and the rail had cut into the longitudinal stringer considerably, mashing the fibers, so that the amplitude of this advance wave was very great and very noticeable, and the force it exerted was sufficient, as the spikes had been driven down hard, to wear away the flanges of the rail under and on each side of the spike heads to an appreciable depth below the general plane of the flange surface. It should be stated that trains were not allowed to run at a speed of more than 15 miles per hour over this bridge.

Analyzing the data given by the author, it appears that at Bethayres and Pleasantville, where the creeping is the greatest, the tracks are in the one case on an embankment on low, swampy ground, and in the other on an embankment on the low-lying salt marshes of the coast; in both cases the compression of the embankment and under-

Mr. Tatnall. lying marsh by the heavy traffic would be relatively great and the wave motion of great amplitude.

The author quotes Mr. Camp as to the effect of temperature on the creeping of rails, and temperature is undoubtedly the second important factor. The creeping of the rails in hot weather toward the old-fashioned and now obsolete stub switch, is well known, as well as the creeping toward the free ends of the rails at a drawbridge; and, undoubtedly, many of the movements noted in these observations are the result of temperature changes, particularly the west-bound track at Hopewell and the north-bound track at North Wales and Gywnedd, as well as one or two other points, where whatever movement has been noticed is in a direction opposite to the traffic. It is to be regretted that the distance and direction to the nearest switch or crossing has not been noted as giving a partial clue to the influence of the temperature on the movement. When the difference between the summer length and the winter length of the two rails immediately adjoining a draw-span, in a bridge about 2 000 ft. long, amounts to 6 or 8 ft., it can readily be believed that the changes in temperature could cause greater backward and forward movements than the fluctuations shown in the tables.

In the writer's opinion, the wave motion, as influenced by the frequency and weight of the traffic, the compressibility of the roadbed, and the underlying natural surface, and the expansion of the rails, under increases of temperature toward an unstayed point in the track, are the only causes of creeping rails, and that such factors as curves, grades, speed of trains, nature of traffic, other than weight, etc., etc., are negligible quantities in the discussion. This opinion seems to be upheld by the data presented.

Vol. XXX. OCTOBER, 1904. No. 8.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THEORY AND FORMULAS FOR THE ANALYTICAL COMPUTATION OF A THREE-SPAN SUSPENSION BRIDGE WITH BRACED CABLE.

Dicussion.*

By Irving P. Church, Assoc. Am. Soc. C. E.

Irving P. Church, Assoc. Am. Soc. C. E. (by letter).—It is not without some diffidence that the writer presents this discussion on Mr. Moisseiff's paper, from the fact that some of the features which seem to him to be at variance with the principles of mechanics may involve errors of small practical importance and thus be fairly negligible; and that he has been unable to find time to ascertain by numerical trial whether or not such is the case. Mr. Church.

The first result which the writer is unable to confirm is on page 576, in the paragraph beginning with "If the two chords of the truss, etc.," and ending with Equation 8. Here the statement is made that a load on the bridge would be sustained by the two chords (now supposed suspended without any diagonal stiffening members between) in proportion to their vertical deflection under the stress caused. Mathematical work follows, and, finally, claim is made for the truth

* This discussion (of the paper by Leon S. Moisseiff, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for August, 1904), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Communications on this subject received prior to November 25th, 1904, will be published subsequently.

Mr. Church. of Equation 8, but without demonstration. This paragraph seemed so obscure to the writer that he has attempted to throw light on it, in his own mind, by working out the following simple case:

In Fig. 3, $O\ B_1$ and $B_1\ C$ are two elastic bars, of equal length, l_1, and sectional area, A_1, jointed to each other at B_1; their upper extremities being supported on fixed pins, O and C. $O\ B_2$ and $B_2\ C$ are two other bars, but of greater length, l_2, and of different sectional area, A_2, pivoted to each other at B_2 and to the fixed pins, O and C.

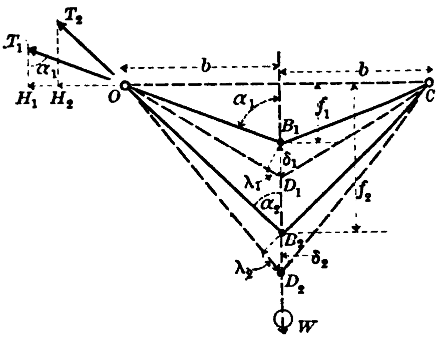

FIG. 3.

These "chords," of two rods each, sustain a weight, W (although one chord would be sufficient), suspended on a single vertical suspension rod (dotted in the figure). The suspension rod is supposed to have been fitted originally to the pins at B_1 and B_2 in such a way that all four bars are in tension when the load has settled into place. Let the vertical deflection, $B_1\ D_1$, of the pin, B_1, be denoted by δ_1 and $B_2\ D_2$, that of the pin, B_2, by δ_2. Let f_1 and f_2 represent the respective "versed sines" of the two chords, and b the common half-span. These are marked in the figure, as also the angles, α_1 and α_2. Let T_1 and T_2 denote the respective total tensile stresses induced in the bars, $O\ B_1$ and $O\ B_2$. The values of T_1 and T_2, and also those of their horizontal components, H_1 and H_2, will depend on the two vertical deflections, and upon other quantities, in accordance with the following relations (δ_1 and δ_2 are much exaggerated in the figure):

Since the extremity, B_1, has sunk to a new position, D_1, vertically underneath B_1, and since the extremity, O, is considered fixed, the elongation of the bar has a value, λ_1, which may be obtained by projecting $B_1\ D_1$, or δ_1, upon the line, $D_1\ O$; that is,

$$\lambda_1 = \delta_1 \cos.\ \alpha_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

But if E is the modulus of elasticity of the material of the four bars (the same for all), we have, from a familiar relation in the "Mechanics of Materials,"

$$\lambda_1 = \frac{T_1\ l_1}{A_1\ E};$$

and hence Equation 38 becomes

$$T_1 = \frac{E\ A_1\ \delta_1 \cos.\ \alpha_1}{l_1} \ldots\ldots\ldots\ldots\ldots\ldots (39)$$

Now $H_1 = T_1 \sin.\ \alpha_1$; hence, with $\frac{f}{l_1}$ for $\cos.\ \alpha_1$, and $\frac{b}{l_1}$ for $\sin.\ \alpha_1$, we have Mr. Church.

$$H_1 = \frac{E\,A_1\,\delta_1 f_1\,b}{l^3_1} \qquad (40)$$

Similarly, for the bar, or member, $O\,B_2$, in its elongated condition under stress,

$$H_2 = \frac{E\,A_2\,\delta_2 f_2\,b}{l^3_2} \qquad (41)$$

whence, by division, writing L_1 for $2\,l_1$, and L_2 for $2\,l_2$ (L_1 and L_2 being thus the total lengths of the two "chords," respectively),

$$\frac{H_1}{H_2} = \frac{A_1\,L^3_2 f_1\,\delta_1}{A_2\,L^3_1 f_2\,\delta_2} \qquad (42)$$

It is also easily seen from Equation 39 that

$$\frac{T_1}{T_2} = \frac{A_1\,L^2_2 f_1\,\delta_1}{A_2\,L^2_1 f_2\,\delta_2} \qquad (43)$$

If the suspension rod is fitted accurately to the pins, B_1 and B_2, before loading, all five rods being then straight but under no stress, then, after loading, on account of the stretching of the part, $B_1\,B_2$, of the suspension rod, δ_2, will be greater than δ_1.

But if the elongation of the part, $B_1\,B_2$, of the suspension rod is considered negligible under the circumstances, $\delta_1 = \delta_2$, and Equation 42 reduces to

$$\frac{H_1}{H_2} = \frac{A_1\,L^3_2\,f_1}{A_2\,L^3_1\,f_2} \qquad (44)$$

If, now, the proportion, $\frac{L_2}{L_1} = \frac{f_2}{f_1}$, were true, we should immediately, from Equation 44, derive the relation stated by the author in his Equation 8, *viz.*,

$$\frac{H_1}{H_2} = \frac{A_1\,L_2 f_2}{A_2\,L_1 f_1} \qquad (8)$$

But such a proportion ($L_2 : L_1 :: f_2 : f_1$) is evidently very far from the truth when the versed sine is small compared with the length of the chord or cable, as is the case in the spans of most suspension bridges. The writer, therefore, is forced to the conclusion that Equation 8 is not even approximately correct (though it is possible that the author had in mind some different relation between δ_1 and δ_2 than that of equality).

If the pins, O and C, move horizontally to a slight extent during the gradual loading of the structure in Fig. 3, the mathematical outcome, of course, is more complicated, the amount of such movement being partly dependent on the design of the "side-span cables."

If the author had presented the detail of treatment of a simple specific case, like that in Fig. 3, his meaning, doubtless, would have been clearer, and the degree of approximation more evident.

Next take up the method of expressing the stresses in the "cables," or chords, when the stiffening truss is in action (middle of page 577).

Mr. Church. In Fig. 4, let $O\ k$ represent a portion of the main cable (*i. e.*, the upper chord of the truss), extending from the tower-pin at O to any segment or "member," k. The forces acting are: The tension, T_o, in the segment next to the tower-pin (its horizontal component can be denoted by H_1, or $m\ H$); the tension in the bar, or segment, k, *viz.*,

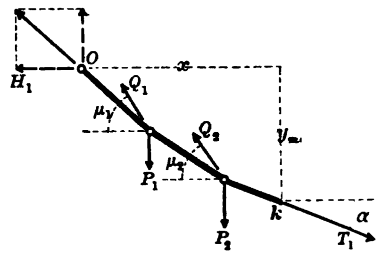

FIG. 4.

T, the value of which is to be expressed; certain vertical forces, P_1, P_2, etc., at the intervening joints; and also certain oblique pulls or thrusts at these joints, coming from diagonal members which are in action, *viz.*, Q_1, Q_2, etc. Summing the horizontal components of the forces for the equilibrium of this portion of the upper chord (no longer a cable under purely vertical loads), we obtain

$$T \cos. \alpha = H_1 + Q_1 \cos. \mu_1 + Q_2 \cos. \mu_2 \ldots\ldots\ldots\ldots (45)$$

or,

$$T = m\ H \sec. \alpha + (Q_1 \cos. \mu_1 + Q_2 \cos. \mu_2) \sec. \alpha \ldots\ldots (46)$$

instead of the result obtained by the author, *viz.*, $T = m\ H \sec. \alpha$; which result, therefore, would seem to be contrary to the laws of mechanics.

A similar treatment of a portion of the main-span stiffening chord gives a similar outcome. To assign to this lower chord the properties of a cable under purely vertical loads (*i. e.*, to claim that the stress in any part is equal to $n\ H \sec. \alpha$) seems quite erroneous; since the stresses existing in the diagonal members are of great importance in the treatment of this lower chord, which is not supposed to be under any stress at all (aside from that due to its own weight), unless the diagonal members are called into action (by a moving load or change of temperature), in which case we find an oblique Q at each joint.

As regards forming an expression for the "bending moment" for any cross-section of the stiffening truss, when regarded approximately as a curved beam, or inverted arch rib, it should be remembered that, in the case of a beam or rib, where the forces in a section are equivalent, not to a stress couple and a shear simply (as in a simple beam), but to a couple, a shear, and a thrust, the moment of the couple (*i. e.*, the bending moment) is not equal to the algebraic sum of the moments of the forces acting on the portion of the beam on one side of the cross-section, unless the axis of moments is taken through the center of gravity of the plane figure formed by the sections of the two chord members of the truss (or very nearly so).

Thus, in Fig. 5, we have a representation, as a "free body," of a portion of the stiffening truss situated between the tower pin, O (the pressure on which is replaced by its two components, horizontal component, H, and vertical component, R), and a cross-section made near the middle of any panel. In the three members of the truss thus cut (*viz.*, k, r, and v), we find acting the stresses, T_1 (in upper chord), Q (in the diagonal member), and T_2 (in the lower chord). The point, n, of the cross-section is taken at the center of gravity (in side view) of the plane figure formed by the sections of the two chord members. Mr. Church.

The sum of the moments of the forces on the left, taken about n, will be the value of M, or "bending moment," to be used for the purpose for which M would be used when a treatment involving the analogy of a curved beam is adopted. On this basis, we derive (see Fig. 5 for symbols)

$$M = R\,x - P_1\,a - H\,w. \quad \ldots\ldots (47)$$

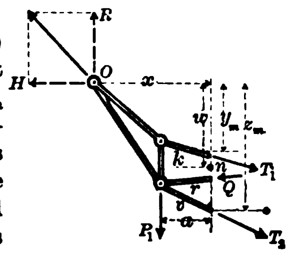

Fig. 5.

The writer is unable to see any warrant for replacing the term, $H\,w$, by $m\,H\,y_m + n\,H\,z_m$, as the author has done in Equation 11; even supposing that correct values of m and n are available. It is indeed true that the product, $H_1\,y_m$ (*i. e.*, $m\,H\,y_m$) would occur in forming moments for the forces in Fig. 4 about the point, k, of the cable or upper chord. But the body shown in that figure, being a portion of the upper chord, is not a stiff beam, and the bending moment in the section, k, is zero.

The writer finds it difficult to recognize any necessity for the invention and use of the artificial quantities, m and n of this paper. A straightforward application of the Principle of Least Work to the entire assemblage of bars or members would seem to answer every purpose; since the stress in each member is capable of being expressed in terms of known quantities and the one unknown, H.

There seems to be in the paper no specific declaration of the important feature that the tower-pins have free horizontal movement; though it is implied, of course, in the evident assumption that the "H" is the same for all three spans.

Vol. XXX. OCTOBER, 1904. No. 8.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

A RATIONAL FORM OF STIFFENED SUSPENSION BRIDGE.

Discussion.*

BY MESSRS. W. HILDENBRAND, JOSEPH MAYER, R. S. BUCK and W. W. CREHORE.

Mr. Hildenbrand.

W. HILDENBRAND, M. AM. SOC. C. E.—Looking at Mr. Lindenthal's paper in the light of a "study," similar to the paper read by the late George S. Morison, Past-President, Am. Soc. C. E., some eight years ago,† the speaker would place it in the same line with the latter, as a work of merit and interest. The design described possesses features which are worth examining and which may find practical application in the construction of suspension bridges.

The foremost feature, as indicated in the title of the paper, is the form of stiffening construction, in the shape of an inverted braced arch. This system has often been applied to smaller bridges, but, the speaker agrees with Mr. Lindenthal, it may also be adapted with advantage to large bridges, though he does not agree with him that it is the only rational stiffening construction, as the paper seems to indicate. Mr. Lindenthal is overestimating the merits of his design. by attributing to it too many good qualities.

As a special feature of the design, it is pointed out by the author that the height of the stiffening construction is greatest at the points

* This discussion (of the paper by Gustav Lindenthal, M. Am. Soc. C. E., printed in *Proceedings* for August, 1904), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.
Communications on this subject received prior to November 25th, 1904, will be published subsequently.

† *Transactions*, Am. Soc. C. E., Vol, XXXVI, p. 359.

of the greatest moments. The same feature is contained in the Point Bridge, at Pittsburg, built in 1875, which, altogether, represents precisely the same theoretical principles as the present design. Mr. Hildenbrand.

It is certainly good construction, to give a stiffening truss the greatest height at the right place, but it is doubtful whether it is economical in this case, because the bottom chord (or at the Point Bridge, the top chord) runs to the top of the towers. As it would be very injudicious to transfer the wind pressure to the top of a slender column, special wind chords become necessary, which, as seen from the strain diagram, are almost as heavy as the bottom chords themselves.

In regard to the artistic effect, the drawings and pictures do not convey a complete idea. They give merely the outlines, which are pleasing, and show the appearance of the bridge from a great distance, but the speaker is sure that the large, latticed, box-shaped, bottom chords and posts running high up in the air, will be an unpleasant disturbance to the eye of an observer who is on the bridge itself. The beauty connected with an open suspension bridge will be lost, and this bridge will not differ in appearance from that of a truss or cantilever bridge.

Evidently, there are reasons why all engineers do not believe in the superiority of the braced-arch system. This is demonstrated by examples of suspension bridges recently built. The speaker will not refer to the Williamsburgh Bridge, because it has wire cables, but the new Elizabeth Bridge, at Buda Pest, may be mentioned, as the author himself has often quoted it as a model of modern engineering. This bridge has eye-bar chains and other features in common with the Lindenthal design, but it is made rigid with stiffening trusses, and not by the braced-arch system.

Of American engineers, the speaker will quote the opinion of Mansfield Merriman, M. Am. Soc. C E., who was a member of the board of engineers called upon to examine the possibility and feasibility of executing Mr. Lindenthal's plans for the Manhattan Bridge. In his book, "Higher Structures" published in 1898, Professor Merriman concludes a paragraph on stiffening systems of suspension bridges with the following words:

> "The suspension truss with center hinge is more rational and more effective than one with unbroken chords. If these laws of development hold good for very long spans, the system of bracing the cables, and thus introducing complexity, does not seem to be in the right direction."

Another American engineer, Joseph Mayer, M. Am. Soc. C. E., whose reputation as a mathematician is well established, states that he made a comparison* between a stiffening truss and an inverted braced

* "The Stiffening System of Long-Span Suspension Bridges for Railway Trains," *Transactions*, Am. Soc. C. E., Vol. XLVIII, p. 871.

Mr. Hildenbrand. arch for a railroad bridge of 3 000 ft. span, and found, by calculating separate strain sheets, that the braced cable would weigh over 5 000 lb. per lin. ft. more, than if stiffened by trusses. He says:

"In view of the practical impossibility of calculating accurately the braced-chain design, and of adjusting it as assumed in the calculations, the unit stresses ought to be taken at least 5% less in this design than in the design with three-hinged suspended trusses, if the same degree of safety is aimed at.

"This would make the difference in economy between the two designs still larger."

These examples show that distinguished engineers in America and Europe prefer other stiffening methods than the braced-cable system.

The strongest argument, if correct, against the usefulness of this system, is furnished by Mr. Lindenthal in the following words of his paper:

"The only recommendation of stiffening trusses is that of a convenient construction independent of the cable, which can be made more or less efficient or costly as may suit the fancy of the designer, or the financial resources of the owners, frequently on the principle, 'that half a loaf is better than no bread.'

"From a scientific point of view, the stiffening truss is a crude device, as compared with other modes of stiffening, which do not permit of the same laxity in dimensioning."

In other words, Mr. Lindenthal states that stiffening trusses can be constructed with any desired degree of rigidity or flexibility and cost, but that the braced arch has but one rigidity, one dimension, and one cost. The speaker does not share this view, that the braced arch could not be proportioned for various flexibilities; but, if this were true, the braced cable would be the most impractical of all stiffening methods, because, rationally, it could only be used in rare and isolated cases for railway bridges, and would be prohibitory for the construction of highway bridges.

A hungry man (using the author's parallel), who is wise, takes half a loaf if he cannot get a whole one, and if the inverted braced arch represents the whole loaf, it cannot readily be sold, as there are too few people who are able to pay for it.

What, for instance, would have become of the intercourse between Brooklyn and New York if the people of those cities had not accepted the Brooklyn Bridge, which, in Mr. Lindenthal's estimation, seems to possess only the value of a quarter loaf? On the other hand, have the people fared badly because they did accept it? The answer to the first question is that there would be no bridge to-day, but plenty of annoyance; and the answer to the second question is that the people have fared well, and are the owners of a large bridge which, every year, carries 100 000 000 passengers safely and comfortably without a single person, excepting Mr. Lindenthal, ever knowing or noticing, that the bridge is not "scientifically stiff."

Mr. Hildenbrand.

This leads to the question of rigidity of suspension bridges.

It appears from Mr. Lindenthal's paper that the chief aim of his design is extreme rigidity, which he considers the paramount quality of a suspension bridge. Even strength and economy are subordinate to stiffness. Bridges with less stiffness than is attributed to his design are styled: limber, undulating, improperly or unscientifically designed, inferior, "makeshifts."

Some of these epithets may be applicable to some old bridges, where the speed of travel used to be restricted, but Mr. Lindenthal extends them also to bridges where such restriction does not exist, simply because their rigidity does not reach the degree which he considers standard.

The question is, what is the standard? There is none; but the speaker would like to ask, which of the following two suggestions seems to be the more practical and sensible: One specifying that a bridge should conform to the theory and formulas of a certain professor, with strict limits as to the deflection under enormous loads, regardless of cost, or one specifying simply that the bridge must be capable of carrying its intended traffic with perfect safety and comfort to travelers, regardless of the amount of deflection, but not to exceed a certain cost? It is not absolutely necessary that these two specifications must be separated; but, if they are, the speaker would select the second as a scientifically practical design, which would be chosen by capitalists, while the former would have only mathematical merit, and might be chosen by theorists.

Referring again to the Brooklyn Bridge, the speaker would ask, is there anyone who was ever prevented from crossing, or who was inconvenienced while crossing that bridge on account of insufficient rigidity of the floor? Has anyone ever heard or read of any complaint that the flexibility of the bridge had restricted or retarded traffic? The speaker has never heard of such complaint, and claims that the Brooklyn Bridge is practically and scientifically stiff, notwithstanding occasional statements to the contrary by people who have never taken the trouble to obtain the proper information.

Mr. Lindenthal states that if the tracks of the middle span were loaded with cars and the side spans unloaded, that the stiffening trusses would be wrecked, if not the whole structure. The speaker would like to see this statement proved by exact calculations before answering it. He knows that it is not true, if the load does not much exceed 2 000 lb. per lin. ft., for which the bridge was calculated. Many unproved assertions are hastily made and carelessly repeated which do injustice to a structure and its builder or builders. It is but just to the latter that such statements are either proved or retracted.

Both Mr. Lindenthal and Mr. Moisseiff describe correctly the dif-

Mr. Hildenbrand.

ference between an upright arch and a catenary or inverted arch, *viz.*, that the former is in unstable and the latter in stable equilibrium. Mr. Lindenthal continues:

"To that condition many badly designed suspension bridges owe their life; it covers a multitude of sins against good engineering."

Mr. Moisseiff thinks that it is "inducive to the sound sleep of the engineer." The speaker draws exactly the opposite conclusions. If a suspension bridge owes its life to the stable equilibrium of the cable, the engineer who built it committed no sin if he took advantage of that good quality of a catenary and relied upon it; and his engineering is better and he is more wide-awake than the engineer who throws away the free gifts of Nature, and who, after having seen an upright arch, does not look farther, but, by artificial means, forces the inverted arch into a condition which it already possesses naturally. To do so seems to be as injudicious as constructing a hanger from which a weight is suspended, like a column which supports a weight on top.

The author further says: "Every suspension bridge is theoretically an invertod arch bridge." This may be true, but, practically, there is a great difference between the two, because an arch bridge must be almost absolutely stiff or it collapses, while a suspension bridge can be absolutely limber without collapsing.

The speaker does not wish to be misunderstood as arguing or advocating that a suspension bridge should be without any rigidity; this is not at all his view, as he has shown practically in the bridges he has built. To be serviceable, a bridge should, and must, have sufficient rigidity, but the speaker believes in the judiciousness of varying the rigidity according to the demands and conditions of each case. Mr. Lindenthal, on the other side, urges the greatest possible rigidity in all cases, and a bridge without it, though it may be sufficiently stiff, is, in his view, only a "makeshift." The extreme to which Mr. Lindenthal goes is best shown in his own words:

"The merit * * * of a bridge structure, * * * can be gauged by no other criterion so reliably as by the degree of rigidity."

"A suspension structure, to be comparable with other bridge systems, * * * must be dimensioned on the same conditions of strength and stability. When that is done its vaunted economy vanishes."

The last sentence is perfectly true, but, if the principle expressed in the preceding sentence is carried out, not only the economy will vanish, but the suspension bridge itself will go out of existence. Mr. Lindenthal has gone as far as to publish* pictures showing how the Manhattan Bridge, if built on his plan, would look if placed upside down, compared with the Williamsburgh Bridge if reversed. He actually used these pictures as an argument for the superiority of his

* *Engineering News*, October 1st, 1908.

design, because the inverted Manhattan Bridge would stand while the inverted Williamsburgh Bridge would collapse! Mr. Hildenbrand.

What is the advantage of such excessive rigidity? There is absolutely none. Is anything or anybody benefited by it? No. Bridges are not built to be put upside down or to satisfy certain abstract analytical formulas. They are for practical use, and a bridge which is safe and answers fully its intended purpose, and, also, is built at the least expense, is more of a scientific structure than one which can boast of nothing but excessive rigidity.

To sum up: The speaker will repeat that Mr. Lindenthal's paper, theoretically and constructively, is a meritorious contribution to suspension bridge literature, for which much credit is due him. If the paper had been confined to a decription of the design in which his theories and detailed constructions were exemplified, the speaker's discussion, if presented at all, would have been short. Mr. Lindenthal, however, has connected with his paper statements referring to other engineering structures and to special engineering principles, which are the cause of this more lengthy discussion.

First, it must be inferred from the paper that the inverted braced arch is the best, if not the only rational, stiffening system. The speaker has shown that distinguished engineers of America and Europe hold different opinions.

Second, the speaker has argued that bridges built on different systems and different principles are not "makeshifts," as they are styled by Mr. Lindenthal, and that the Brooklyn Bridge, which is mentioned specially in the paper, fulfils its purpose and all the conditions for which it was designed.

Third, Mr. Lindenthal has attempted to establish a standard for the rigidity of suspension bridges, and the speaker has shown that this standard is based on arbitrary and unscientific assumptions; that it is uselessly severe and extravagant, and that it leads to absurdities if carried to the extreme.

Probably the essence of the whole of Mr. Lindenthal's paper may be comprised in this: That a suspension bridge, in order to be properly designed should have the highest attainable rigidity; while the essence of the speaker's whole discussion may be expressed in the words that a suspension bridge, if properly designed, should have the greatest possible flexibility which is compatible with safety and the practical purpose of the bridge.

Joseph Mayer, M. Am. Soc. C. E.—The title of this paper suggests that previous forms of stiffened suspension bridges are irrational, and the body of the paper contains various assertions condemning other forms which have been developed by experience and which are justified mainly by two fundamental facts which have controlled the growth of suspension-bridge design in the past and will do so in the Mr. Mayer.

Mr. Mayer. future. These two facts are the superior strength of steel wire and the difficulty of making satisfactory connections between a wire cable and the web members of a stiffening truss. The independent stiffening truss has been a necessary consequence of these two facts. The expense of wirework limits the economic usefulness of suspension bridges to long spans.

Highway bridges of more than 800 ft. span, and railway bridges of more than 1 300 ft. span, can, under favorable conditions, when rock anchorages are available, be built economically of the suspension type. The speaker designed a cantilever and a suspension bridge across Sydney Harbor with a span of about 1 300 ft., for a double-track railway and a 60-ft. highway. The bids submitted for both designs were nearly the same. Natural rock anchorages were available. Subsequently, he designed another suspension bridge for the same site. The design was found to be by far the lightest of many cantilever and suspension-bridge designs submitted. The specifications for both kinds of bridges were substantially identical. For much less than 1 300 ft. span, a railway suspension bridge is uneconomical and cannot compete with a cantilever bridge. At Quebec the chief engineer in charge preferred the cantilever type, and stated that a suspension bridge would only be considered if there was a difference in cost of at least $500 000 in its favor. Wire cables of more than 200 000 lb. ultimate strength and 180 000 lb. elastic limit per square inch cost erected at most two and one-half times as much per pound as eye-bar cables made of nickel steel of one-quarter the elastic limit. The bar heads and pins of the latter add about 25% to the weight of continuous bars of the same strength. This shows that wire cables cost only one-half and weigh about one-fifth as much as eye-bar cables of the same strength. The difference in weight becomes of extreme importance for very long spans.

A suspension bridge of three spans made of eye-bar cables is inferior in economy to a cantilever bridge. The type of bridge recommended in the paper, therefore, could only claim superiority on account of its appearance. The pictures published of the design proposed are deceptive because they do not show the cross-bracing between the stiffening trusses, which will detract greatly from the graceful appearance secured by the use of cables without bracing.

A cantilever bridge can be built on substantially the same outlines as the bridge proposed by the author; it is much stiffer, which, in the opinion of the author, gives it superiority, and it is far cheaper and looks equally well.

To abandon wire cables means practically to abandon all the advantages of a suspension bridge. Any suspension bridge without wire cables, therefore, can hardly claim to be called pre-eminently rational. One of the great advantages of the suspension-bridge type of structure is the possibility of building a perfectly safe bridge which will show very little deformation under the usual highway-bridge loads, and which will avoid excessive stresses under partial loads extending over either one-half of the main span or one or two of three spans. This advantage can be secured by the use of shallow stiffening trusses which are independent of the cables. Mr. Mayer.

Such partial loads may never occur during the whole lifetime of the bridge. A deflection of several feet at the center or the quarter of one of the spans, produced by such imaginary distribution of the live load is no disadvantage whatever. The high degree of stiffness insisted upon by the author is absolutely useless for a highway bridge or for a bridge with so many independent units of load as will use any East River bridge not intended for heavy steam-railway trains.

The serviceableness of a bridge is not proportional to its rigidity. A certain degree of rigidity is needed; anything beyond is useless. An 80-ft. plate girder has a much greater deflection than an 80-ft. truss bridge of equal strength; the former is preferred by most, in spite of it.

The degree of rigidity required is very different for a bridge used by heavy railway trains of half the length of the main span and for a highway bridge of many roadways subject mostly to approximately uniform loads. For the former, a three-span suspension bridge, with the side spans of half the length of the main span, is very unfavorable, unless the end spans are held in the center.

In most situations the three-span suspension bridge, for railway purposes, is an uneconomical type. The shallow stiffening truss, which is the most suitable for highway suspension bridges, gives too large deflections in a railway bridge. The stiffening truss which reduces the deflections just sufficiently for the needs of a railway bridge allows so little deformation of the cables that it alone has to carry the larger part of the moments and shears produced by partial loads. A much deeper stiffening truss than required for stiffness then becomes economical, for securing the required strength. The depths of the stiffening truss given by the author are nearly those suitable for a railway bridge.

The author's assertions in regard to the relative size of the temperature stresses in hinged and continuous stiffening trusses are totally in error, as the writer has shown in his discussion with him.* A deep stiffening truss must be hinged in the center and at the ends of the main span. In this case the temperature stresses become small. For the writer's design of 2 800 ft. span, of the Hudson River Bridge, they amounted to less than 4% of the moving-load stresses. Without hinges, a deep stiffening truss is uneconomical on account of the excessive temperature stresses.

* *Engineering News*, November and December, 1901.

Mr. Mayer. The most economical depth for a railway suspension bridge stiffening truss, hinged at the center and ends, is about one-eighteenth of the span. This depth is needed at the center of the half span. The economic general dimensions of each half span are about those of an independent span of the total length of the half span of the suspension bridge.

It is easy to design a center hinge allowing vertical motion and yet preserving the continuity of the bottom chords, which serve as chords of the lateral truss; the drawback of a center hinge mentioned by the author, therefore, does not exist.

When the span is very long and heavy the center hinge is best designed so that there is a hinge in the horizontal wind truss as well as in the vertical trusses. The lateral truss of each half span is then separate. The cables take, in this case, without excessive lateral deflection, the wind pressures transferred by the wind trusses to the center of the main span. The wind stresses of the top lateral system, which come to the center of the span, must be taken down to the bottom chords by means of overhead cross-bracing with brackets. The weight of the lateral system is greatly reduced by this arrangement, since each lateral truss is only half as long as the main span. This arrangement was adopted in the writer's design of the Hudson River Bridge, and was approved by a commission of eminent engineers.

The author's design is an attempt to revive an almost defunct type of bridge which is much inferior in economy to a properly designed wire cable suspension or a cantilever bridge, and much inferior in looks to a suspension bridge with a shallow stiffening truss offering abundant stiffness for highway traffic.

One feature deserving high praise in the design of the author is the hinged tower. Such a tower, however, harmonizes much better with light unbraced cables than with heavy braced trusses. It has been adopted in the revised design of the cable suspension bridge for the same site, and is a great improvement on all earlier designs.

Mr. Buck. R. S. Buck, M. Am. Soc. C. E.—The spandrel-braced suspension bridge is a most interesting study, with claims to merit chiefly due to the degree of rigidity furnished by the depth of trussing at the quarters, which, for certain service and conditions, is inadequately furnished by the familiar type of suspension bridge, with dip of cable and depth of truss of economical proportions.

Much interest is added to the subject by the exhaustive mathematical treatment to which it has been subjected in the two supplementary papers by Mr. Moisseiff and Mr. Cilley.* But the "broad policy, of giving the city the benefit of the advice, criticism and expert knowledge of specialists outside of the department," to which Mr.

* *Proceedings*, Am. Soc. C. E., for August, 1904, pp. 514 and 568.

Cilley refers (p. 515), was not as prolific of recorded results in all the phases of this case as in the mathematical treatment. Mr. Buck.

It may be unfortunate that bridge design and construction are not more exclusively a matter of mathematical calculation; but it is a condition which cannot yet be escaped that the controlling considerations in gauging the rationality of a bridge design are based upon actual experience, rather than upon the attenuating processes of the higher mathematics. While mathematics forms an essential part of the foundation of bridge designing, it has serious limitations, most of which can only be supplied by a rational study of accomplished works.

While there will probably be no disposition to question the correctness of the two mathematical demonstrations, especially as they are mutually confirmatory and not antagonistic, adequate knowledge of the controlling physical and economic conditions, as well as theories of suspension bridge design, construction and maintenance, precludes acceptance of the author's implied conclusion, that the familiar type of wire-cable bridge is hopelessly wrong in principle, as well as thus far imperfect in its applications, and that the spandrel-braced eye-bar chain type is the "rational" solution of all the difficulties and objections found in the other.

The present high development of the art of bridge construction is due rather to the gradual improvement in the works of the many along familiar lines than to the bold conceptions of the few on lines radically new. No type of bridge can attain a high degree of excellence in one or two applications. In such complex problems several applications are necessary to develop all the difficulties and prove their correct solutions.

While the spandrel-braced chain bridge may in time become as superior as the author now considers it, its superiority can hardly be considered as proved in the light of accomplished facts; no more than in the same light it can be proved that the controlling principles of the wire-cable suspension bridge are unsound and inadequate.

The chief claims made for the "rational form of stiffened suspension bridge" are

1.—Greater rigidity and consequent general superiority;
2.—Superiority of details, due chiefly to pin connections;
3.—Greater ease and speed of manufacture and erection;
4.—Greater economy, chiefly in the eye-bar chains, as compared with wire-cables.

1.—The greater rigidity of the spandrel-braced chain bridge, as compared with the wire-cable bridge with a stiffening truss of practicable depth, is, as a matter of theoretical mechanics, clearly apparent, but the universal value and certainty of proper action of this rigidity are not yet demonstrated either theoretically or practically.

Mr. Buck. It should be remembered that the stresses in a stiffened suspension bridge of any form are more or less indeterminate under varying conditions of loads and temperature. This indetermination cannot be entirely removed by mathematical treatment, nor by the degree of accuracy of adjustment practicable in bridge construction.

When a high degree of rigidity is imposed upon an indeterminate structure, the risk of serious consequence, because of departure from assumed and theoretical conditions, becomes materially greater than in a structure where there is enough flexibility to permit any overloaded member to pass part of the overload along to others and obtain relief.

In the deep spandrel bracing with long panels, light sections with constant reversals in the stiffening members and with adjustable diagonals, there cannot be the same freedom from serious indeterminate stresses that there is in the familiar form of stiffening truss of moderate depth and minimum number of adjustable members. In the rigid attachment of comparatively light bracing to a long, broad and massive chain, supported independently of the bracing, maintenance of assumed conditions within proper limits is not assured.

The author states (p. 553) that:

"All the older suspension bridges are limber and undulating structures, without adequate provision against deformation under moving loads."

While the stiffening systems of the older suspension bridges have not proved adequate to meet the excessive and unanticipated demands on their strength and endurance, there are very few, if any, cases where failure has occurred, except when the loading was far beyond that provided for. Wherever inadequacy has developed or failure occurred it has been due to imperfect structural details or to imperfect knowledge, in designing, of the actual conditions of stress created by the loading, not to any inherent or incurable weakness in the principle of design.

The author's illustrative comparison between the Forth Bridge and the Brooklyn Bridge (p. 554) is not quite clear as to point, especially as the Forth Bridge was duly designed to carry the load which he states it will carry, while the Brooklyn Bridge was not designed to carry the load which he states would wreck it.

The failures of the stiffening system of the Brooklyn Bridge were not due to the lack of rigidity, but primarily to the improper loading to which it was subjected, and secondarily to the interference with its uniform deflection by the overfloor stays, coupled with the inadequate form of section of the bottom chord. These are conditions and details of construction readily remedied, and not at all inseparable from the general form of design.

In suspension-bridge design, as in all engineering work, the most valuable lessons are to be learned from a rational study of failures and their causes. There is serious insecurity in speculation that indiscriminately consigns to the junk heap accomplished works, which, though imperfect, are valuable in that they have served often far beyond the original purpose of the design, and have marked by actual performance sound and unsound features of design. Mr. Buck.

The author states (p. 556):

"The merit and serviceableness of a bridge structure, other things being equal, can be gauged by no other criterion so reliably as by the degree of its rigidity."

Again (same page):

"Absence of sufficient rigidity marks the inferior bridge."

"Rigidity" is a term often misapplied to metal structures and confused with "Stability" and "Strength."

"Sufficient rigidity" is the condition in a bridge where there is no overstraining of parts, where the motion of parts under deflection does not occasion wear, and where the deflection is not so great as to impede traffic. These conditions can be fully met in the familiar type of suspension bridge, and at the same time permit of much greater deflection than is possible with safety in a simple truss, arch, cantilever, or spandrel-braced chain bridge.

The matter of deflections in suspension bridges was fully covered in the investigations and discussions of the North River Bridge project, whereby, as the author suggests, "Much of the fog that surrounds the theory of rigidity was cleared away."

Major Raymond, in referring to the conclusions of the Board of Army Engineers appointed to investigate this project, wrote as follows: *

"The Army Board devoted considerable attention to this question. It remarked that 'the great distinction between the stable equilibrium of a suspension bridge, which cannot break down from the failure of any stiffening member, and the unstable equilibrium of a truss, arch or cantilever bridge, in which the failure of a member may involve the collapse of the entire bridge, ought to receive full recognition in the adoption of unit stresses and safety factors.' Again, the Board remarked that 'rigidity is in this case of much less importance than it is in most other kinds of bridges; indeed, it may be shown that a certain small flexibility is a positive advantage in suspension bridges;' and still again, 'the Board does not doubt that within narrow limits a certain degree of flexibility is an advantage to the bridge. Deflections in a system of stable equilibrium do not impair the safety of the structure, as they do in an unstable system like the upright arch, and they may exert a very beneficial influence in modifying the dynamic effects of a rapidly varying live load.'"

George S. Morison, Past-President, Am. Soc. C. E., wrote as follows: †

"A long span suspension bridge necessarily changes its shape with every change of load, and changes, too, in such manner as to

* *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 459.

† *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 367.

Mr. Buck. relieve local strains, every unstiffened suspension bridge having some shape of perfect equilibrium for every possible loading. These changes of shape play an important part in proportioning a suspension bridge, and so long as they are kept within limits which do not disturb convenience of operation, they are a source of strength instead of weakness. A suspension bridge must be permitted to change its shape within proper elastic limits, and this change of shape must be made the basis of calculations in proportioning the structure."

The foregoing conclusions have long remained unquestioned, and are sound principles of design, established by practice as well as by theory.

"Sufficient rigidity" is not a fixed function in suspension bridges for all classes of service. This must be much greater for the 1 800-ft. railroad bridge at Quebec than for the 1 470-ft. city highway bridge required in the case of the Manhattan Bridge. In the former case, the stresses in the stiffening system, due to the heavy trains it must carry, will be great and frequent, and the tendency to excessive deflection will be present with every passing train. In the latter, the loadings will rarely depart so greatly from uniformity as to cause considerable stiffening-truss stresses or material deflections. No loading that could possibly occur in the legitimate use of the bridge could cause serious stresses or objectionable deflections.

In this case, with ample strength in cables and suspenders, high unit stresses can be allowed in the stiffening truss with perfect safety, and the maximum deflections under assumed possible conditions of loading can be regarded as of only theoretical interest.

The speaker fully agrees with the author in his conclusion (p. 554) that:

"A suspension structure, to be comparable with other bridge systems, therefore, must be dimensioned on the same conditions of strength and stability."

In fact, all suspension bridges are stronger and more stable than other forms of construction, not only on account of being in stable equilibrium, but because they are generally built with a greater factor of safety in the most vital parts—cables and suspenders.

There is far more strength and greater security, in the thoroughly tried material and high safety factors of the wire cables and suspenders, than can be supplied by the increased rigidity due to deep spandrel bracing.

The minimum elastic limit of the steel wire to be used in the cables of the Manhattan Bridge is 180 000 lb. per sq. in. The allowed stress under maximum working load is 60 000 lb. per sq. in. The factor of safety is 3.

The minimum elastic limit of the nickel-steel in eye-bars specified for the same bridge is 48 000 lb. per sq. in. The allowed stress under maximum working load is 30 000 lb. per sq. in. The factor of safety is 1.6. Therefore, the strength of the cables, based on elastic limits, is almost twice as great as that of the chains. Mr. Buck.

While the uncertainty, which now surrounds the successful and economical manufacture of 18 by 2-in. nickel-steel eye-bars, may in time be removed, it has not yet been removed, and the likelihood of the existence of a defective component unit, and the injury therefrom, are many times greater in a chain than in a cable. This cannot be compensated by rigidity.

The author states (p. 556) that:

"From a scientific point of view, the stiffening truss is a crude device, as compared with other modes of stiffening, which do not permit of the same laxity in dimensioning."

He then proceeds to demonstrate this from the evolution of the arch, on the hypothesis that "every suspension bridge is theoretically an inverted arch bridge." The analogy between the arch and the suspension bridge is extremely narrow, and is confined to the fact that the cable of the suspension bridge and the rib of the arch produce horizontal reactions and are both treated in calculations as force polygons. In possibilities of span, in construction, in instability of equilibrium and consequent necessity for greater rigidity, the arch is wholly removed from the suspension bridge, and is an extremely inadequate basis for rational conclusions regarding the latter.

The author's "principal difference"—stable equilibrium in the one case and unstable equilibrium in the other—completely destroys the practical value of the analogy.

2.—The author states (p. 562) that:

"The advantages of pin connections with the bracing and suspenders, and of manufacturing and erecting the superstructure as a connected whole, are too obvious for extended description."

It is hoped that the author will ultimately furnish the "extended description" withheld, as the merits of pin connections are not recognized as universally as he infers.

Apparently, there has been some change in the author's views since he stated, of the study of a North River bridge design, by Mr. George S. Morison, that: "* * * * Another good feature of his stiffening truss is the riveted connections, with web members riveted up also at their intersections. It greatly helps to diffuse the bending strains throughout the frame, and the so-called secondary strains at all the connections and intersections are a decided gain on the side not only of greater stiffness, but greater safety, contrary to the received theoretical considerations on this subject."

"The received theoretical considerations on the subject" are presumably the prejudices in favor of pin connections.

In bridge practice, pin connections have been greatly superseded by riveted connections.

* *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 444.

Mr. Buck. It has been found that pins wear seriously when members assembled in them are subject to frequent reversals. Especially is this the case in suspension bridges where the diagonal members of the stiffening system are adjustable rods. This feature is the most serious element of weakness and deterioration in the Brooklyn Bridge, not its lack of rigidity. It was a serious element of weakness in the Niagara Railway Suspension Bridge. In both cases, the constant shifting of the rods on the pins caused wear and loss of adjustment. On the Brooklyn Bridge there has been quite extensive renewal of pins and diagonal bars.

In the designs covered by the author's paper, there must inevitably be the same shifting of the diagonals on the pins with every passing load. While the wearing action in this case, of course, would not be as great as in the Brooklyn and Niagara Bridges, it must exist. The narrow limits of deflection of a spandrel-braced chain bridge would not prevent this wear, as the amount of distortion necessary to cause the shifting is very small. Should it be attempted to put such initial stress on the diagonals that they could not shift on the pins, except under unusual loads, there would be likelihood of serious overstressing due to temperature changes, as well as to serious indetermination in the stresses.

This is evidently recognized by the author, as he states (p. 560):

"It should be remarked that in this system it is preferable that the cable assume its proper equilibrium curve under full dead load, and that the diagonals, which are without strain at middle temperature, be made adjustable in length. The diagonals are strained only from live load and temperature changes."

In the later designs of wire-cable suspension bridges, the use of adjustable members and pins is reduced to a minimum, the trusses being riveted throughout. In such trusses, flexure can take place without wear, and, on account of their shallowness, deflection can be much greater, without injurious effect, than can occur under the heaviest possible traffic.

The advantage of the so-called "positive attachment" of the suspenders to the chains by pins, over the attachment to the cables by bands, is not apparent. The older forms of cable bands, despite their crudeness and the weak grip they have on the cables, have proved remarkably efficient, and have given very little trouble. They sometimes slip when first put on, but it is a simple matter to check this and restore adjustment.

A cable band can be provided with almost any degree of grip on the cable, which will be as positive as can possibly be required, even by the demands of suspender loads greater than have yet been provided for.

Fig. 8 shows the cable bands of the Brooklyn Bridge, the Williamsburgh Bridge and the Manhattan Bridge.

It can hardly be questioned that, as the first has proved reasonably efficient and satisfactory, the others are amply so. Mr. Buck.

In the cable design, the floor system is connected with the stiffening system in such a manner that the latter distributes the moving panel loads and their impact effects over a number of suspenders. In the chain design, the suspenders are interposed between the floor beams and the stiffening trusses, and, without any relief from the trusses, must receive the full dynamic effect of the moving loads. In other words, the suspenders are merely more or less lengthened floor-beam hangers—a form of construction now avoided when possible.

In the cable design, it is an added element of security, as compared with the chain design, to have more points of attachment between the suspended structure and the cables. In the case of the Manhattan Bridge, these are about 43 ft. apart in the chain design and 18 ft. apart in the cable design. The more numerous the suspenders, the less serious the effect of a defective unit, and the better the distribution of the load into the cables.

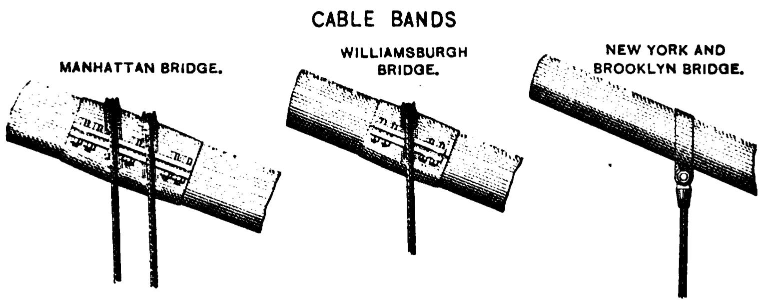

FIG. 8.

3.—It is difficult to make a positive comparison between the eye-bar chain and wire cable, in the matter of time and cost of construction, as what can be accomplished in the former case is wholly a matter of conjecture.

Previous to the construction of the cables of the Williamsburgh Bridge, it was quite generally supposed that wire-cable construction must necessarily consume a serious length of time.

In this case, the time actually consumed in stringing the wires of the cables was seven months, including time lost on account of weather, the work being carried on during the winter.

There were, of course, many serious delays, but causes for the greater part of these were wholly foreign to the method of construction; while those for which it was responsible are now, in the light of the experience acquired, capable of being reduced materially or avoided altogether.

Mr. Buck. Causes for delay invariably attend novel methods of construction in greater numbers, where difficulties must first be developed by actual performance and then the means of surmounting them be worked out.

In cable construction, all the essential problems—both economical and structural—pertaining to the manufacture of material and erection have been solved.

In the construction of eye-bar chains of the size proposed by the author, most of the essential problems pertaining to commercial manufacture of material and erection must yet be solved, whether the chains be made of wire links or nickel-steel eye-bars.

In the case of the chains, the temporary bridge required for erection must be many times stronger and heavier than would be required in the construction of the cables, and, while threading 18-in. eye-bars weighing 3½ or 4 tons on 18-in. pins, under distinctly difficult conditions, may be economical and expeditious, in the absence of explicit demonstration to that effect, skepticism is at least pardonable.

4.—The author, in the course of his paper, makes several statements regarding the economy of features of the spandrel-braced chain design, and lack of economy of the wire-cable design, from which it is natural to infer that he wishes to convey the impression that the former as a whole is distinctly more economical than the latter.

While this point is not capable of conclusive and specific settlement at the present time, on account of the lack of adequate information on which to base the cost of a spandrel-braced chain bridge of the proportions of either the Manhattan or the Quebec Bridge, there are certain established facts which tend strongly to prevent the formation of this impression.

The relative cost of wire cables and so-called equivalent eye-bar chains may be, from known conditions and contract prices, so far deduced as to show that the chains must cost very much more than the cables.

The contract price of the cables of the Williamsburgh Bridge was a little less than 14¼ cents per lb., erected.

The contract price of the nickel-steel eye-bars for the Blackwell's Island Bridge was 8.03 cents per lb., and of the nickel-steel pins 10.03 cents per lb., erected. These bars were to be 16 by 2-in., with 38-in. heads.

On the one hand, the price of the Williamsburgh Bridge cables was doubtless higher than could be obtained now, because many difficulties originally anticipated in procuring the specified grade of wire, and in attaining economical and reasonably rapid construction in building the cables, have been now satisfactorily solved. Further, prices of all such work were very much higher at the time this contract was let than have prevailed for the past year.

On the other hand, the cost of manufacture of the 18 by 2-in. eye-bars with 44-in. heads proposed is wholly unknown, as no such eye-bars have yet been made, and there are doubts as to the practicability of making them, except at a prohibitory cost. Mr. Buck.

Further, the difficulties and cost of erection are equally unknown. However, in view of the fact that elaborate special plant must be provided for the erection of the chains, which will be chargeable to these alone, while in the Blackwell's Island Bridge the same erection plant will handle and support the eye-bars which will handle and support the remainder of the work, the conclusion is fully warranted that the erection cost in the former case will be materially greater than in the latter.

In view of the known contract prices and the other conditions named above, it appears almost impossible that, at the present time or in the near future, the unit prices of wire cables and nickel-steel chains would be farther apart than 14 to 10, respectively, which, because of the chains being two and one-quarter times as heavy as the cables, would make the cost of the former, on this basis, 60% greater than the latter. There is warrant for the belief that the difference would be still greater.

The chains and cables are but single items of cost in the respective designs, but they are the items wherein material advantage in point of economy has been claimed for the chains.

A complete and conclusive comparison of cost of the two designs is not possible, in the absence of complete detailed plans of the chain design and fuller development of the actual conditions of manufacture and construction involved. However, there are no grounds whereon to base the claim that, as a whole, the chain design is more economical than the cable design.

To summarize: It does not appear that in either of the cases cited by the author—the Manhattan Bridge or the Quebec Bridge—is there any substantial evidence that the spandrel-braced chain design possesses adequate grounds for preference over the other designs proposed for the same cases.

If the bridge is for varied city traffic, where, whatever the weight of the aggregate loadings, material departure from uniformity is rare, and where appearance must count for so much, the rational design for long spans still appears to be, in the light of the best available evidence, the wire-cable suspension bridge.

If the bridge is for heavy railroad traffic, where departure from uniformity of loading is great and frequent, and where, therefore, a high degree of rigidity is essential, the rational design for long spans still appears in the same light as the cantilever, until the length of span approaches 2 000 ft., when the wire-cable suspension bridge again appears as a strong competitor, even for the heaviest class of railroad service.

Mr. Crehore. W. W. Crehore, M. Am. Soc. C. E.—One point that the speaker would like to emphasize is that this question seems to turn on the degree of rigidity it is sought to obtain. Most engineers have had experience with smaller structures than these under consideration, and it will be pretty hard to convince any engineer who has had experience with railroad and similar bridges that rigidity is not a good thing. Most engineers believe that rigidity is a good thing; that rigidity is something which prevents abnormal wear and tear in a bridge; that is, of two bridges of the same span and for the same load, that which is the more rigid will last the longer, and, therefore, is more economical from that point of view, if not from any other.

If rigidity is a good thing in a truss bridge, why should not rigidity be a good thing in a suspension bridge? The same laws of mechanics govern in a suspension bridge that govern in a truss bridge. Will not the distortion of a structure always cause some wear and tear which should be avoided as much as possible?

The point to be brought out is this: A suspension bridge takes a fixed position from its dead load. The amount of distortion caused by bringing on a live load depends upon two things, (1) the effectiveness of the stiffening system, and (2) the ratio of the live load to the dead load per linear foot. In other words, the inertia of the structure's dead weight prevents some distortion from taking place, and the stiffening system prevents some—the remainder takes place.

Of two bridges constructed alike, if a heavier live load is put on one than is put upon the other, it is expected that the former, in passing across the bridge, will produce the greater distortion. Now, the longer the span, the greater must be the dead load to carry the same live load; and, of two bridges constructed of the same span, that which must carry a heavier live load must have the greater dead weight. The amount of vibration or distortion, therefore, varies with the ratio of the live load per linear foot to the dead load per linear foot, other things being equal.

A suspension span across the North River would be about 3 000 ft. long, and one across the East River would be considerably less than 2 000 ft. long. Probably a sufficient live load for modern railway traffic over a North River span would be that assumed in Mr. Morison's design, *viz.*, about one-fourth of the dead load; whereas the emergency load which ought to be provided for any East River span being built to-day is about one-half of the dead load. The traffic for which the old Brooklyn Bridge was designed was one-fourth of the dead load; and, with such a traffic, that bridge is comparable to a bridge carrying the modern railway traffic over the North River. In the speaker's opinion, the gentlemen who have have discussed this paper have made the mistake of comparing a bridge for modern traffic over the East River with a bridge of the same span to carry the traffic of twenty years ago. The two are not comparable, except when the live lôad (the only part of the load which causes distortion) is the same part of the total load in each case. Mr. Crehore.

The modern East River traffic must be understood to include some provision for the future, such as heavily loaded automobile trucks massed close together, and an increase in weight of the railroad rolling stock. An emergency loading of this sort concentrated in one short stretch of roadway would amount to about one-half of the dead load of the bridge per linear foot; and, while this would have no injurious effect upon the cables of a suspended cable structure, it would cause excessive distortion, injurious to the stiffening system, unless the stiffening trusses were unusually deep. Here, again, the fact must be met that, when the stiffening trusses are of too great a depth, excessive loading will react injuriously on the cables, preventing their assuming an easy or natural position (as it were), and causing undue local strains. It works around in a circle, showing that when the limit of deflection is assumed to be fixed and must not be exceeded, the feasibility of preventing further distortion by means of stiffening trusses is limited, and the process cannot be continued indefinitely by adding to the depth or to the material of the stiffening trusses. To be really effective, the dead weight of the cables themselves would need to be increased so as to provide sufficient inertia to absorb a considerable part of the excessive loading before distortion should begin. Such a method of increasing the dead weight, however, would be very costly and is prohibitory on the ground of wasteful expenditure as long as any other method is known whereby sufficient rigidity may be obtained more cheaply.

Now, Mr. Lindenthal's paper illustrates this truth. It brings out the fact that the braced suspension design will furnish the required degree of rigidity under the excessive loading referred to, that is, where the live load is about one-half the dead load. This result is accomplished by putting the principal dead weight of the bridge into the cables, and practically dispensing with the stiffening trusses entirely; or, better, incorporating the stiffening and the cables together in one system. If it were attempted to obtain the same amount of rigidity in a suspended-cable design as can be obtained in the braced-cable design—the live load per linear foot being one-half the dead load in both cases—it would be necessary to use stiffening trusses of such great depth that the design would be at once condemned on account of its disproportionate and ungainly appearance; and, on the other hand, the people of New York City do not want any more bridges on which the police are required to regulate the traffic to prevent possible congestion injurious to the structure.

Regarding those passages quoted by Mr. R. S. Buck from the *Transactions* of this Society, in which Mr. Morison and Major Ray-

Mr. Crehore. mond express opinions on different designs for a 3 000-ft. span across the North River, the speaker understands them to refer to cases where the traffic load per linear foot would not exceed one-fourth of the dead load per linear foot. When it is proposed to bridge spans of from 500 to 1 000 ft., where the live load is as much as or more than one-half the dead load, a suspension bridge design is usually discarded as inappropriate and of insufficient rigidity, or as requiring too great an expenditure to make it rigid. Why should not this ratio of live to dead load receive the same consideration on longer spans? In his admirable paper, just referred to, Mr. Morison states that the first thing to do in designing a suspension bridge is to establish the limit within which deflection may be permitted, and then to design to it. This deflection limit, however, depends somewhat upon the ratio of the live to the dead load, as well as upon the material and the shape of the structure; and all of Mr. Morison's subsequent discussion of the subject leads to the conclusion that there is a practical limit to the use of a suspended-cable bridge, and that this limit is reached when stiffening trusses cannot be designed sufficient to restrict the amount of deflection permissible by the nature of the traffic.

Granting, then, that such a limit exists for the suspended-cable bridge, and that the author's braced-cable design furnishes a method of supplying the necessary rigidity far beyond that limit, it seems to the speaker that this discussion should be upon the question of where the economy of the suspended-cable design leaves off and that of the author's design begins, rather than upon the simple question of whether or not rigidity is a desirable quality in bridges of any kind.

Vol. XXX. OCTOBER, 1904. No. 8.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE INSTALLATION OF A PNEUMATIC PUMPING PLANT.

Discussion.*

By Elmo G. Harris, M. Am. Soc. C. E.

Elmo G. Harris, M. Am. Soc. C. E. (by letter).—Mr. Diamant has well presented the extraordinary conditions under which this particular pump must act, and the many difficulties accompanying its erection. It may be of interest to describe briefly the special features involved in the operation of this system of pumping and give some of the mathematics involved in proportioning a plant properly. Mr. Harris.

The general principles involved are easily stated and readily understood. The special features are:

First.—The system, once charged with air, is closed to the atmosphere; the one charge being forced alternately into one tank while being drawn out of the other. Hence the energy of expansion (or compression) in the air is not lost, as in the common forms of direct air-pressure pumps. The only recognizable losses in the system, outside the air compressor, are: Expansion in the low-pressure air-pipe immediately after switching, friction in the air-pipes, conduction of heat out of the air-pipes (or absorption, which is a gain), and leakage of air. These are not capable of formulation—except the first and second, and these only approximately.

* This discussion (of the paper by Arthur H. Diamant, Jun. Am. Soc. C. E., printed in *Proceedings* for May, 1904), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

Communications on this subject received prior to November 25th, 1904, will be published subsequently.

Mr. Harris. *Second.*—The automatic switching device, by which the air is alternately exhausted from one tank and delivered to the other. This is accomplished by utilizing the difference of pressure inside and outside the tank from which air is being exhausted. For convenience, the mechanism utilizing this is placed in the compressor room. If the tanks are near the free surface of the water supply, there will, at the time the switch should act, be suction in the air-pipe leading to the tank from which air is being exhausted, and which is filling with water. This suction, of course, extends throughout the air-pipe, and is utilized in the compressor room, in conjunction with free air pressure, to operate the switch.

In case the tanks are deeply submerged, as in the pump under discussion, suction will not occur, but the tanks will fill under pressure. In this case the pressure outside the tank is communicated to the compressor room by a "dip" pipe, which descends to the tank level, and through which air, in minute quantity, is forced continuously, thus registering within the compressor room the head outside the tanks, while the main air-pipe registers the pressure within the tanks; as before said, when the pressure within the tank is less than outside, the switch acts. The switch is made adjustable; that is, it can be made to act at any desired difference of pressure. Evidently, the details of the switch can be varied without limit. When suction occurs before the switch acts (which is usually the case) the leakage is replaced automatically by an adjustable valve placed outside a check-valve opening into the intake pipe between the switch and the compressor.

Another method of operating the switch is by a mechanism which acts at a prescribed number of strokes of the compressor —the number being that necessary to complete a cycle.

It may be remarked, before going further, that the submergence of the tanks is not necessary.

With the development of this system of pumping, many problems have been presented for solution, some purely mechanical, while others require a mathematical analysis. The latter have proved very interesting and instructive.

In the process of such analysis,

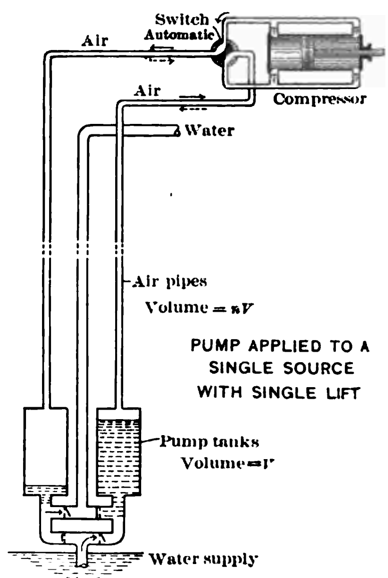

FIG. 6.

it will be necessary to use the following symbols. Though the analysis may be considered intricate, the final formulas are unexpectedly simple and easy of application: Mr. Harris.

Let P_0 = Delivery pressure—a constant—in pounds per square inch;

P_1 = Pressure throughout the system immediately after switching;

P_x = Pressure of air entering a compressor— a variable;

V = Volume of one pump tank—a constant—in cubic feet;

V_y = Volume of air in delivering tank at pressure, P_0 —a variable;

$n\ V$ = Volume of one air-pipe;

p_1 = Pressure at which water begins to enter tank from which air is being exhausted;

p_0 = Lowest pressure reached (this occurs just before switching);

q_a = Effective volume, intake of compressor, in cubic feet per second;

q_w = Average water delivery, in cubic feet per second;

Q = Total volume taken into compressor, while working pressure down from P_1 to p_1, or approximately P_0 to p_1 in any case and approximately P_0 to p_0 when tanks are near surface of water supply;

R_0 = ratio $\frac{P_0}{p_0}$;

R_1 = ratio $\frac{P_1}{p_1}$.

All pressures are "absolute," that is, gauge pressure + 14.7 lb.

Compressor Capacity ($= q_a$).—The first problem is to find the necessary intake capacity of the compressor. In this, fortunately, the problems of work and temperature inside the compressor need not be considered, and, therefore, in the analysis, the temperature of the air may be considered as constant, though it will be necessary, finally, to apply a coefficient to provide for the effect of expansion due to the heating of the air as it passes through the hot intake valves.

Assume that a small volume, $d\ Q$, of air at the pressure, P_x, is taken out of the exhausting tank and forced into the delivery tank, where the pressure is P_0, and its volume is $d\ V_y$, then, by the law that the pressure multiplied by the volume is constant:

$$P_x\, d\, Q = P_0\, d\, V_y; \text{ or } d\, Q = \frac{P_0}{P_x}\, d\, V_y \ldots\ldots\ldots\ldots\ldots \mathrm{I}$$

Also, by the same law, the sum of the product of the pressure multiplied by the volume must be constant, since the quantity (or mass) of air in the system does not change. When one tank is full of water, and its air-pipe is full of air at the pressure, p_0, the other tank and

Mr. Harris. air-pipe must be full of air at the pressure, P_o. Under this condition, the sum of the products is

$$P_o\, V(1+n) + p_o\, V\, n.$$

At any other time the sum of the products is

$$P_x\, V(1+n) + P_o\,(V_y + n\, V).$$

Hence, $P_o\, V(1+n) + p_o\, n\, V = P_x\, V(1+n) + P_o\,(V_y + n\, V)$.II

To simplify, put $p_o = \frac{P_o}{R_o}$ and Equation II reduces to

$$\frac{P_o}{P_x} = \frac{V(1+n)}{V\left(1+\frac{n}{R_o}\right) - V_y} \qquad \text{III}$$

Substitute Equation III in Equation I, and

$$d\,Q = V(1+n)\,\frac{d\,V_y}{V\left(1+\frac{n}{R_o}\right) - V_y}.$$

Integrating between the limits, $V_y = V_1$ and $V_y =$ zero, there results:

$$Q = V(1+n)\log_e \frac{V\left(1+\frac{n}{R_o}\right)}{V\left(1+\frac{n}{R_o}\right) - V_1} \qquad \text{IV}$$

Let V_1 represent the volume of air in the delivery, or high-pressure tank, when water begins to enter the other; that is, when the pressure in the other tank has dropped to p_1; this marks a change in the operation; see Fig. 7. Just at this period there must be enough air, at the pressure, p_1, in the volume, $V(1+n)$, to fill the space, $V - V_1$, at the pressure, P_o, in the other tank, and its own air-pipe at the pressure p_o. Hence the equation:

$$p_1\, V(1+n) = P_o\,(V - V_1) + p_o\, n\, V \qquad \text{V}$$

or,

$$P_o\, V_1 = V\,[P_o - p_1 + n\,(p_o - p_1)].$$

Now, n is a fraction, and p_o and p_1 are small and nearly equal, in practice; hence $n\,(p_o - p_1)$ can be neglected. Then:

$$V_1 = \frac{V}{P_o}\,(P_o - p_1) \qquad \text{VI}$$

Putting Equation VI in Equation IV, there results:

$$Q = V(1+n)\log_e \left[\frac{1+\frac{n}{R_o}}{1+\frac{n}{R_o} - \frac{P_o\, p_1}{P_o}}\right]$$

$$= V(1+n)\log_e \left(\frac{1}{1 - \frac{P_o - p_1}{P_o + n\, p_o}}\right)$$

putting $\frac{P_o}{p_o}$ in place of R_o.

Mr. Harris.

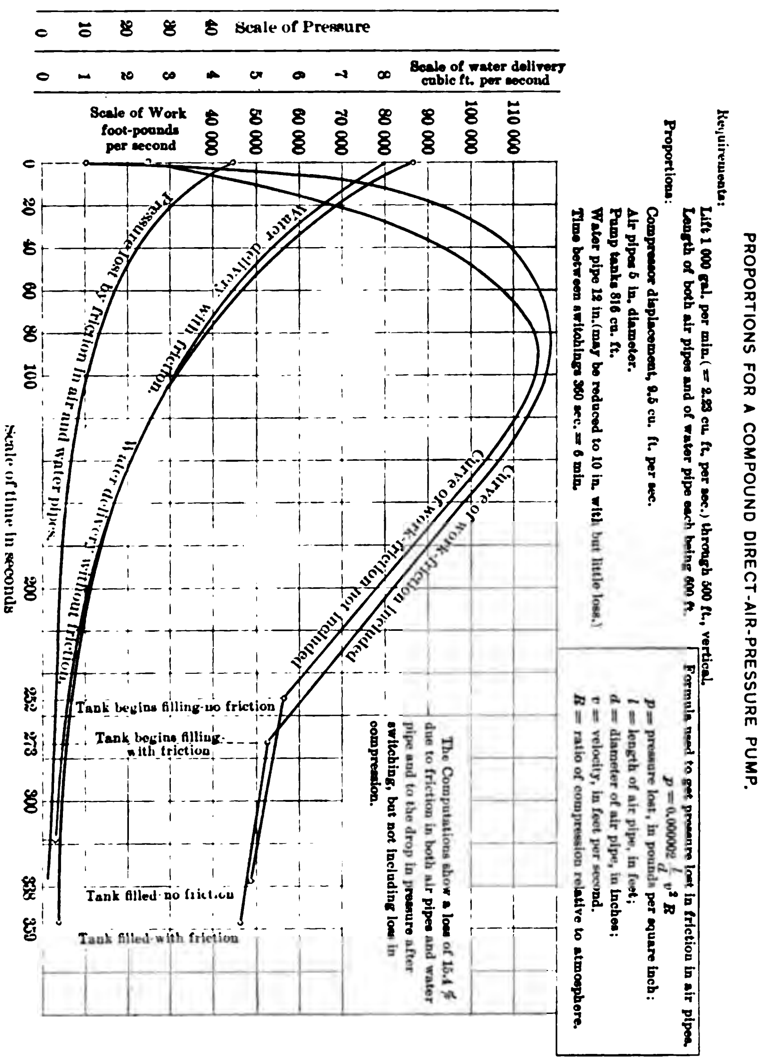

FIG. 7.

Mr. Harris. Now, as before stated, $n\, p_o$ will be quite small, as compared with P_o, and it can be neglected, if desired, to simplify the formulas. Equation VI would then become:

$$Q = V(1+n)\log._e \frac{P_o}{p_1} \ldots\ldots\ldots\ldots\ldots\ldots \text{VII}$$

This gives a simple formula for Q, the volume taken into the compressor while reducing the pressure from P_o to p_1 (in a tank full of air). To be precise, it should now be noticed that the operation begins properly with a pressure, P_1, somewhat less than P_o. This is due to the expansion into the low-pressure pipes just after switching. This pressure, P_1, can be found readily by the condition of the constancy of the sums of the products of the volumes by the pressures. Thus, equating the sums just before and after switching, there results:

$$P_1\,(V + 2\,n) = P_o\,V(1+n) + p_o\,n\,V$$

or,

$$P_1 = \frac{P_o\,(1+n) + n\,p''}{1 + 2\,n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{VIII}$$

P_1, thus found, would be put in place of P_o in Equation VII.

The effect of friction in the air-pipe between the tank and the compressor must now be considered.

When the pressure of the intake of the compressor is P_x, that in the tank from which the air is drawn will be greater by the amount lost in friction while passing through the pipe. The equation for this loss is, in form,

$$f = c\,\frac{l}{d}\,V^2\,R$$

where c is an experimental coefficient. From the best experimental data obtainable, it is found to be about 0.000002, when

f = lost pressure, in pounds per square inch;
l = length of pipe, in feet;
d = diameter of pipe, in inches;
v = velocity of air in pipe, in feet per second;
R = ratio of compression, in atmospheres.

In many rules for computing the loss by friction, the factor, R, is erroneously omitted. In this case, $R = \frac{P_x}{14.7}$, and, therefore, is variable, but in any installation all are constant in the formula except P_x. Then, for simplicity, let

$$\frac{0.000002}{14.7}\,\frac{l}{d}\,V^2 = k \ldots\ldots\ldots\ldots\ldots\ldots \text{IX}$$

Then the lost pressure would be $k\,P_x$, and, in Equation II, $P_x\,(1+k)$ should be put in place of P_x, but this will in no way change the process by which Equation VII is derived. With this change. Equation VII becomes

$$Q = V(1+n)\times(1+k)\log.\frac{P_1}{p_1}.$$

If the compressor takes in a volume, q_a, per second, the time consumed in working the pressure down from P_1 to p_1 is Mr. Harris.

$$t_1 = \frac{Q}{q_i} = \frac{V}{q_a}(1+n)(1+k)\log.\frac{P_1}{p_1}.$$

During the remainder of the time in one cycle, the water is flowing into the tank, following up the air, and keeping it at nearly constant pressure (when the height of the tank is only a few feet); in other words, for every cubic foot of air taken out, a cubic foot of water flows in. Hence, evidently, the time consumed in this last period of the cycle is

$$t_2 = \frac{V}{q_a}$$

and the total time, $T = t_1 + t_2 = \frac{V}{q_a} + \frac{V}{q_a}(1+n)(1+k)\log.\frac{P_1}{p_1}.$

If q_w is the average rate of delivery of the water, evidently,

$$q_w = \frac{V}{T}$$

Whence, $$q_a = q_w\left[1 + (1+n)(1+k)\log.\frac{P_1}{p_1}\right] \ldots\ldots\ldots\ldots\ldots \mathrm{X}$$

which is the desired equation.

In practice, k should not exceed 0.1, and will usually be less. If great precision is to be attempted, Equation X must be solved by a tentative process, for k is a function of q_a. k may be first assumed as 0.1, to get an approximate value of q_a, whence V in Equation IX, and a closer value of k. This will be sufficiently close for practice.

It is probably useless to attempt extreme precision in these computations, on account of temperature changes which cannot be formulated. Hence, as a safe and simple working formula the following may be used:

$$q_a = q_w\left[1 + 1.1\,(1+n)\log.\frac{P_1}{p_0}\right] \ldots\ldots\ldots\ldots\ldots \mathrm{X}a$$

p_0 will commonly be near atmospheric pressure (or 150), that is, when the tanks are near the surface of the water, but it may be greater or less, according to whether the tanks are submerged or placed above the water. Inspection of Equation X reveals the fact that the greater p_1 is, the less will be q_a. For this reason, there is an advantage in having the tanks submerged.

Evidently, if the air is heated by contact with hot surfaces while entering the compressor, the effective intake capacity is reduced. To allow for this circumstance, q_a, as above computed, should be multiplied by $\frac{\tau_2}{\tau_1}$, where τ_1 and τ_2 are the absolute temperatures before and after entering the compressor, respectively.

Maximum Rate of Work.—The compressor capacity having been determined, the next problem in the design of a plant is to find the

Mr. Harris. maximum rate of work for which provision must be made in the steam end of the compressor. The nature of this problem can best be presented by first studying the case of isothermal compression. In this the well-known formula for work, using the symbols heretofore applied, is

$$\text{Work per second} = P_x\, q_a \times \log. \frac{P_o}{P_x} \ldots\ldots\ldots\ldots \text{XI}$$

In this, P_x is variable, and, evidently, the work will be zero when P_x = zero, and again, when $P_x = P_o$ (since log. $1 = 0$), and, by the method of calculus, it is found to be a maximum when log. $\frac{P_o}{P_x} = 1$; that is, when $\frac{P_o}{P_x} = 2.72$.

Note that hyperbolic logarithms must be used in all the foregoing equations as they appear. If common logarithms are to be used, multiply by 2.3.

Inserting the condition for a maximum in Equation XI and reducing to foot-pounds per second, there results:

$$\text{Maximum work} = 52.9\, P_o\, q_a.$$

A curve showing the work by Equation XI is given in Fig. 7. In practice, the curve does not reach zero at either end.

To find the maximum work when temperature changes are considered, one must start with the established formula for work when compression is adiabatic, *viz.*:

$$\text{Work} = \frac{n}{n-1} P_x\, q_a \left[\left(\frac{P_o}{P_x}\right)^{\frac{n-1}{n}} - 1 \right] \ldots\ldots \ldots \text{XII}$$

where n is the "temperature exponent" and equals 1.41 when no cooling occurs.

By the methods of the calculus Equation XII will be found to be the maximum when $\left(\frac{P_o}{P_x}\right)^{\frac{n-1}{n}} = n$; or when $P_x = \frac{P_o}{n^{\frac{n}{n-1}}}$.

This, inserted in Equation XII, gives

$$\text{Maximum work} = \frac{P_o\, q_a}{n^{\frac{1}{n-1}}} \ldots\ldots\ldots\ldots\ldots\ldots \text{XIII}$$

When $n = 1.41$ Maximum work $= 62.3\, P_o\, q_a$ foot-pounds per second.
" $n = 1.25$ " " $= 59.0\, P_o\, q_a$ " " " "
" $n = 1.00$ " " $= 52.9\, P_o\, q_a$ " " " "

the last number having been derived by analysis of Equation XI.

As a simple approximate rule, the maximum horse-power rate may be taken as $0.1\, P_o\, q_a$.

This maximum rate should not be confused with the average.

Efficiency.—The only loss of energy chargeable to this system is that caused by the drop in pressure due to expansion into the low-pressure pipe just after switching. This drop is shown in Equation

VIII. The ratio of this change of pressure is $\frac{P_o}{P_1} = \frac{1+2n}{1+n+\frac{p_o}{P_o}n} = r,$ Mr. Harris.

for simplicity. The necessary work to restore this pressure is

$$P_o\ V(1+n)\log. r,$$

while the useful work done during a cycle is $(P_o - 14.7)\ V$; that is, the water displaced multiplied by the gauge pressure. Hence

$$\text{Efficiency} = E = \frac{(P_o - 14.7)\ V}{(P_o - 14.7)\ V + P_o\ V(1+n)\log. r}$$

$$= \frac{1}{1 + \frac{P_o}{P_o - 14.7}(1+n)\log. r} \quad \ldots\ldots\ldots\ldots\ldots \text{XIV}$$

Losses due to heat and friction are not included. It should be noticed that this loss is dependent on n. Its amount is illustrated by the following: E changes but little with other values of P_o and p_o.

$P_o = 100$ $n = 0.1$ 0.2 0.4 0.6 0.8 1.0

p_o = 14.7 $E = 0.91$ 0.85 0.74 0.66 0.60 0.55

Friction Losses.—In the operation of a plant the velocity of the intake pipe will be constant, but the pressure variable, while, in the discharge air-pipe, the pressure will be constant and the velocity variable. According to Equation IX, the loss in the intake is, in pounds per square inch,

$$\frac{0.000002}{14.7}\frac{t}{d}\cdot V^2 P_x = k\ P_x = f_i \quad \ldots\ldots\ldots\ldots\ldots \text{XV}$$

and the loss due to the same air passing through the discharge pipe at the pressure P_o, is $\frac{0.000002}{14.7}\frac{t}{d}\left(\frac{P_x}{P_o}V\right)^2 P_o = k\frac{P^2_x}{P_o} = f_i\frac{1}{R_x}$ XVI

To find the friction losses at intervals in the cycle, or to show such by a curve, assume convenient intervals of time (5 or 10 sec.) which indicate by t_x. Then,

$$t = \frac{Q_x}{q_a} = \frac{v(1+n)(1+k)\log.\frac{P_1}{P_x}}{q_1}.$$

Whence, adapting to common logarithms,

$$\log._{10} P_x = \log._{10} P_1 - \frac{t_x}{\frac{V(1+n)(1+k)}{q_a}}(0\ 434)\ldots\ldots \text{XVII}$$

Thus, tabulate P_x corresponding to t_x and apply the slide-rule to get the friction losses from Equations XV and XVI.

At any time, the rate of water discharge will be

$$w_x = \frac{P_x}{P_o}q_a.$$

This can be tabulated with the other quantities, and the friction loss in the water pipe worked out accordingly by well-known formulas. Curves worked out by the foregoing methods are shown in Fig. 7.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the Volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final Publication.

GEORGE CURTIS TINGLEY, M. Am. Soc. C. E.*

DIED APRIL 30TH, 1904.

George Curtis Tingley was born in Windham, Connecticut, on January 28th, 1831, and died 73 years later, April 30th, 1904, at his home in Providence, Rhode Island.

His early education was obtained at the South Windham Academy, and in 1848 he entered Trinity College, Hartford, graduating with the class of 1852. With the exception of a few years immediately after completing his education, during which time he was engaged partly in teaching school, and partly in mercantile pursuits, his entire life was devoted to his profession of engineering.

He was Assistant Engineer on the location and construction of the Hartford, Providence and Fishkill Railroad, between Hartford and Providence (now a part of the New York, New Haven and Hartford Railway System).

About 1860 he entered the office of the late Samuel B. Cushing, Sr., M. Am. Soc. C. E., who was at that time the leading engineer of Rhode Island, and his connection with this office was maintained, in one capacity or another, for a period of thirty-three years.

For the first few years of his connection with the office, the firm name was Cushing and DeWitt, and upon the retirement of Mr. DeWitt, Mr. Tingley succeeded to his branch of the business, which carried with it the construction of the street railroad, at that time in its early stages of operation, and all the work then being done for the great manufacturing firm of A. and W. Sprague. About 1871 the firm of Cushing and Company was formed, which consisted of S. B. Cushing, the elder, his son, S. B. Cushing, Jr., George C. Tingley, and Col. J. Albert Monroe, all at that time or later Members of the American Society of Civil Engineers. This firm was dissolved by the retirement of Col. Monroe and the subsequent death of the elder Mr. Cushing in 1873, after which time it became S. B. Cushing and Company, composed of the younger Mr. Cushing and Mr. Tingley, and so continued until the death of Mr. Cushing in 1888, when Mr. Tingley conducted the business alone until his retirement from private practice in 1892.

* Memoir prepared by John W. Ellis, W. H. G. Temple, Herbert E. Sherman and Richard H. Tingley, Members, Am. Soc. C. E.

In 1892 Mr. Tingley was chosen Chief Engineer of the Union Railroad (now the Rhode Island Company), of Providence, and retained that position until his retirement from actual business in 1901. Mr. Tingley's connection with the engineering of the Union Railroad was a long and active one, covering a period of more than forty years from the time of the establishment of the first horse car lines in Providence to the date of his retirement from practice.

During the period of Mr. Tingley's long and successful private practice of engineering, many important works were designed and constructed by him, his chief attention being given to heavy foundation and bridge work, and to railway engineering. From his office have graduated a long list of engineers, Elmer L. Corthell, Desmond FitzGerald, Henry W. Parkhurst, Herbert E. Sherman, William H. G. Temple, Richard H. Tingley, all Members of the American Society of Civil Engineers, besides a large number of others, successful engineers, not members of this Society.

Mr. Tingley was married twice. First, in 1853, to Georgianna Sage, of Hartford. Four children survive this marriage: Pauline E. Tingley, of Providence, Richard H. Tingley, of New York, Mrs. Wm. D. Livermore, of Lawrence, Massachusetts, and Ernest DeW. Tingley, of New Haven, Connecticut. His second marriage was with Elizabeth Vaughan Polleys. Mr. Tingley had been a widower for five years prior to his death.

Mr. Tingley's entire life from early manhood to ripe old age was crowded with active responsibilities fully equal to those which fall to the average man. These he met and discharged with the highest ability and integrity, yet in the midst of these crowded duties he still found time for the exercise of a benevolence and charity rarely equalled in its usefulness.

Gentle, noble and kind, he has left the most endearing memories with all who knew him.

Mr. Tingley was elected a Member of the American Society of Civil Engineers on September 6th, 1871.

American Society of Civil Engineers.

OFFICERS FOR 1904.

President, CHARLES HERMANY.

Vice-Presidents.

Term expires January, 1905:	*Term expires January, 1906:*
L. F. G. BOUSCAREN,	F. S. CURTIS,
JAMES D. SCHUYLER.	S. L. F. DEYO.

Secretary, CHARLES WARREN HUNT.

Treasurer, JOSEPH M. KNAP.

Directors.

Term expires January, 1905:	*Term expires January, 1906:*	*Term expires January, 1907:*
RICHARD S. BUCK,	ALFRED CRAVEN,	CHARLES S. GOWEN,
GEORGE H. PEGRAM,	JOSEPH O. OSGOOD,	NELSON P. LEWIS,
WILLIAM J. WILGUS,	GEORGE S. DAVISON,	JOHN W. ELLIS,
WILLIAM JACKSON,	E. C. LEWIS,	GEORGE S. WEBSTER,
EDMUND F. VAN HOESEN,	HUNTER McDONALD.	RALPH MODJESKI,
JAMES L. FRAZIER.	ELWOOD MEAD.	CHARLES D. MARX.

Assistant Secretary, T. J. McMINN.

Standing Committees.

THE PRESIDENT OF THE SOCIETY IS *ex-officio* MEMBER OF ALL COMMITTEES.

On Finance:	*On Publications:*	*On Library:*
S. L. F. DEYO,	GEORGE H. PEGRAM,	NELSON P. LEWIS,
L. F. G. BOUSCAREN,	ALFRED CRAVEN,	WILLIAM JACKSON,
RICHARD S. BUCK,	JOSEPH O. OSGOOD,	E. C. LEWIS,
WILLIAM J. WILGUS,	GEORGE S. DAVISON,	RALPH MODJESKI,
CHARLES S. GOWEN.	HUNTER McDONALD.	CHARLES WARREN HUNT.

Special Committees.

ON UNIFORM TESTS OF CEMENT:—George S. Webster, Richard L. Humphrey, George F. Swain, Alfred Noble, Louis C. Sabin, S. B. Newberry, Clifford Richardson, W. B. W. Howe, F. H. Lewis.

ON RAIL SECTIONS:—L. F. G. Bouscaren, C. W. Buchholz, S. M. Felton, Robert W. Hunt, John D. Isaacs, Richard Montfort, H. G. Prout, Joseph T. Richards, Percival Roberts, Jr., George E. Thackray, Edmund K. Turner, William R. Webster.

ON CONCRETE AND STEEL-CONCRETE:—C. C. Schneider, J. E. Greiner, W. K. Hatt, Olaf Hoff, Robert W. Lesley, J. W. Schaub, Emil Swensson, A. N. Talbot, J. R. Worcester.

The House of the Society is open from 9 A.M. to 10 P.M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - 583 Columbus.
CABLE ADDRESS, - - - - "Ceas, New York."

Vol. XXX. NOVEMBER, 1904. No. 9.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1

PROCEEDINGS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

November 2d, 1904.—The meeting was called to order at 8.45 P. M., Joseph Mayer, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 145 members and 44 guests.

The minutes of the Thirty-Sixth Annual Convention, held in St. Louis, Mo., October 3d to 8th, 1904, and of the Society meeting of October 19th, were approved as printed in the *Proceedings* for October, 1904.

James C. Meem, Assoc. M. Am. Soc. C. E., was expected to address the Society and give a description of "A New Method of Tunneling as Applied to the Construction of the 14 and 15-foot Bay Ridge Sewer," illustrating his remarks with lantern slides, but, unfortunately, he was unable to be present, and the Secretary presented Mr. Meem's description. The subject was discussed by Messrs. Henry R. Asserson and Francis Collingwood.

The Secretary made the following announcements:

The transfer of the following candidates, by the Board of Direction, on November 1st, 1904:

FROM ASSOCIATE MEMBER TO MEMBER.

WILLIAM J. CARTER, Cleveland, Ohio.
EDWIN JOHN FORT, Brooklyn, N. Y.
JOHN HUNTER HANNA, Washington, D. C.
JOSEPH FREDERICK HASSKARL, Philadelphia, Pa.
LEWIS JEROME JOHNSON, Cambridge, Mass.
PAUL ALBERT SEUROT, New York City.
GRATZ BROWN STRICKLER, Baltimore, Md.
AARON HOWELL VAN CLEVE, Niagara Falls, N. Y.
CHARLES PAGE WILLIAMS, Cody, Wyo.
SILAS H. WOODARD, New York City.

FROM ASSOCIATE TO ASSOCIATE MEMBER.

JOHN PARRY JOHNSTON, Chicago, Ill.
HERBERT SEDGWICK WILGUS, Brooklyn, N. Y.

The election of the following candidates, by the Board of Direction, on November 1st, 1904:

AS JUNIORS.

ROBERT LEE ALEXANDER, Mound, Ark.
JULES ROWLEY BREUCHAUD, Croton-on-Hudson, N. Y.
JACOB HERBST BRILLHART, Port Griffith, Pa.
ALFRED THOMAS BROWN, New York City.
THEODORE DELONG COFFIN, Jamaica, N. Y.
DEAN GRAY EDWARDS, New York City.
ROBERT FOLLANSBEE, Malta, Mont.
HERBERT MILLER HALE, New York City.
EUGENE NATHAN HUNTING, Pittsburg, Pa.
ROBERT FAULKNER MOSS, Columbia, Mo.
JOHN PRINCE HAZEN PERRY, Jamaica, N. Y.
WILLIAM HALE PHILLIPS, Berkeley, Cal.
CHARLES SCHULTZ, Hannibal, Mo.
WALTER JAMES SPALDING, Lone Dell, Mo.

David Arnold Starbuck, Troy, N. Y.
Earle Talbot, Coyote, Cal.
Engel Bert Van de Greyn, Chicago, Ill.
Henry Christopher Westover, St. Joseph, Mo.

The Secretary announced that the Annual Convention of 1905 will be held at Cleveland, Ohio, during the last full week in June.

Adjourned.

November 16, 1904.—The meeting was called to order at 8.30 P. M., Vice-President Deyo in the chair; Chas. Warren Hunt, Secretary, and present, also, 135 members and 23 guests.

Mr. Walter C. Parmley, M. Am. Soc. C. E., addressed the Society and gave a description of "The Walworth Sewer, Cleveland, Ohio," illustrating his remarks with lantern slides. The Secretary announced the death of Louis Frederic Gustave Bouscaren—Vice-President, Am. Soc. C. E.; Director, 1881; elected Member April 7th, 1875; died November 6th, 1904.

OF THE BOARD OF DIRECTION.

(Abstract.)

October 6th, 1904.—President Hermany in the Chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Croes, Gowen, McDonald, Modjeski, Moore, Noble and Pegram.

The Secretary presented the following report received from the Secretary of the Nominating Committee:

"St. Louis, Mo., October 15th, 1904.

"Mr. Chas. Warren Hunt,
Secretary, Am. Soc. C. E.

"Dear Sir:

"A meeting of the Nominating Committee of the American Society of Civil Engineers was held yesterday evening, October 4th, at the 'Inside Inn.' There were present:

"Past-Presidents:
J. James R. Croes,
Robert Moore,
Alfred Noble,
"District No. 1—George W. Tillson,
" " 1—Albert Carr,
" " 2—J. P. Snow
" " 2—Fred. Brooks,
" " 3—Edward A. Bond,
" " 4—Richard Khuen,
" " 5—L. E. Chapin,
" " 6—John B. Atkinson.

"Meeting organized with Fred. Brooks, Chairman; Albert Carr, Secretary.

"The following nominations were made:

President, C. C. Schneider.
Vice Presidents, M. S. Holman, Emil Kuichling.
Treasurer, Joseph M. Knap.
Directors, District No. 1—Morris R. Sherrerd,
" " 1—A. L. Bowman,
" " 2—Hezekiah Bissell,
" " 3—Edwin A. Fisher,
" " 3—William B. Landreth,
" " 5—George S. Pierson.

"Respectfully,
"Albert Carr,
"Secretary of Nominating Committee."

Adjourned.

November 1st, 1904.—8.40 P. M.—Vice-President Curtis in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Buck, Craven, Croes, Deyo, Ellis, Gowen, Jackson, N. P. Lewis, Noble, Osgood, Pegram and Webster.

The following resolutions were passed:

Resolved, That no further subscriptions for membership in the International Engineering Congress be received after this date, unless remittances for that purpose have been mailed prior to October 31st, 1904.

Resolved, That the charge for the publications of the International Engineering Congress, to those who are not members of the Congress or of the American Society of Civil Engineers, be made $5 for each volume of the Congress publications, and that all who become members of the Society prior to January 1st, 1904, shall receive the *Proceedings* of the Congress free of charge.

A communication from H. S. Haines, Chairman of the Committee in Charge of the International Engineering Congress, was received and ordered printed in the *Proceedings.**

It was resolved that the next Convention of the Society be held in Cleveland, Ohio, during the last full week in June, 1905.

Applications were considered and other routine business transacted.

Ten Associate Members were transferred to the grade of Member, two Associates were transferred to the grade of Associate Member, and eighteen candidates for Junior were elected.

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*See page 455.

VISIT OF MEMBERS OF THE INSTITUTION OF CIVIL ENGINEERS.

The invitation of the American Society of Civil Engineers to the Institution of Civil Engineers to visit America during 1904, was accepted in December, 1902, and on September 3d, 1904, Sir William White, K. C. B., President of the Institution, and a party of 74 Members, Associate Members, Associates, and Students of that Institution, accompanied by 30 ladies and other guests, a total of 104, sailed from Liverpool and arrived in New York late on Saturday, September 10th.

On Monday, September 12th, nearly all members of the party registered at the Society House.

On **Tuesday, September 13th,** the visitors were received officially by the President at the Society House, and, after luncheon had been served, the party entered the new subway at the 60th Street Station, where, by courtesy of the Interborough Rapid Transit Company, a special train had been provided for the trip to the Park Row Station. The City Hall was visited, and Hon. Geo. B. McClellan, Mayor of New York City, said a few words of welcome. The party then returned to the subway and continued an inspection trip of this new work which was not then open to the public. On the same morning the ladies of the British party, accompanied by some American ladies, took an automobile drive through Central Park and the upper part of Manhattan, and were entertained at luncheon at Claremont on the Hudson, returning, *via* the Riverside Drive, to their hotels.

On **Wednesday, September 14th,** there was an all-day excursion to the Croton Dam now in process of construction. Transportation for the party was furnished by the courtesy of the New York Central and Hudson River Railroad Company, and luncheon was served at the Dam by invitation of the contractors, Messrs. Coleman, Breuchaud and Coleman. This excursion occupied the entire day.

Those who did not go on the above excursion made the trip around Manhattan Island in the steamer *Nassau*, which was furnished for the occasion by the courtesy of the Long Island Railroad. This excursion took place in the afternoon.

Both of these excursions were largely attended, not only by our visitors, but by members of the Society and their ladies.

Thursday, September 15th, was devoted to an all-day excursion to West Point. The party left New York, *via* the Hudson River Day Line boat, transportation for the visitors having been tendered by the Company, and arrived at West Point about 12 o'clock. Carriages were taken to the Cullom Memorial where the entire party was received by

Visit of—THE INSTITUTION OF CIVIL ENGINEERS—as guests of—THE AMERICAN SOCIETY OF CIVIL ENGINEERS—at THE UNITED STATES MILITARY ACADEMY AT WEST POINT, SEPTEMBER 15TH, 1904.

General Albert L. Mills, U. S. A., Commandant, and Mrs. Mills, and, after luncheon had been served, and the various buildings inspected, the Corps of Cadets paraded in honor of the party, and were reviewed by General Mills accompanied by the officers of the societies present. The return to New York was made by special train over the West Shore Railroad. A photograph of the party, taken on the steps of the Cullom Memorial after luncheon, is reproduced herewith.

Friday, September 16th, was devoted to various visits to points of engineering interest. One party visited the Filtration Plant at Little Falls, N. J., another the Brooklyn Navy Yard, and in the afternoon parties were made up, and visited, under guidance of the engineers in charge, the following points of interest: The Interborough Power Houses, Pennsylvania Railroad Shafts, Hudson River Tunnel, East River Bridges, and the New York Central and Hudson River Railroad Terminal Work.

On the evening of this day a banquet in honor of the members of the visiting party was given at Delmonico's at 7.30 P. M. The total attendance at this banquet was 260, and the speeches made on that occasion are printed in the following pages.

The party left New York for Montreal, Canada, at 7.30 A. M., on Monday, September 19th, by special train, kindly furnished to the American Society of Civil Engineers for the use of its guests, by the New York Central and Hudson River Railroad.

The Society is also indebted to the Grand Trunk Railway and to the Wabash Railroad for the courtesy of a special train from Toronto to Chicago, and from Chicago to St. Louis, respectively.

SPEECHES AT THE BANQUET

GIVEN TO THE VISITING MEMBERS

OF THE

INSTITUTION OF CIVIL ENGINEERS

BY

THE AMERICAN SOCIETY OF CIVIL ENGINEERS,

DELMONICO'S, NEW YORK.

September Sixteenth, 1904.

COMMITTEE ON BANQUET.

CHARLES WARREN HUNT, *Chairman;*

CHAS. W. BUCHHOLZ, GEORGE H. PEGRAM,

ALLEN HAZEN, WM. L. SAUNDERS.

WELCOME.

CHARLES HERMANY, PRESIDENT, AM. SOC. C. E.

Gentlemen, Members of the Institution of Civil Engineers, Englishmen and Canadians, Civil Engineers of whatever country or nationality, in the name and in behalf of the American Society of Civil Engineers, I greet you. I welcome you. I extend to you our friendship and hospitality.

We feel honored beyond description by the attestation of this magnificent assemblage of professional men—members of a profession that has scarcely a standing of one hundred years, and that ranks to-day as one of the first.

You, gentlemen, make epochs in history and fill the intervening eras with blessings for mankind. You are the vanguards and pioneers who subdue Nature and promote civilization as no organized body of men ever did in the history of the world. We feel honored in being your hosts, in entertaining you, and in giving you the right hand of friendship in a calling that to-day, in all the civilized world, ranks among the highest. We hope that upon this occasion you feel yourselves in touch with our country's enterprises, our institutions, our ambitions and our destinies, as expressed by the American profession of civil engineering. (Applause.)

We do not say this boastfully, but we say it confidently, feeling assured that in assuming this position we do not go beyond our depth. We will make the port that is destined for this noble voyage that we are engaged in.

Much has been done by us as civil engineers; more remains still to be done. Any man who has the privilege of gazing into the countenances of this assemblage and there perceiving the brain power it exemplifies, the physique it exhibits, and the courage that it has heretofore and will hereafter display, cannot doubt that our destiny is grand, that our objects will be accomplished, namely, making men individually and collectively better and greater.

Those of us who have been engaged for a lifetime in this vocation become daily more imbued with the nobility of its character, with the magnificent ends that it has accomplished, and still has in view. As a factor in the agencies of civilization, you will permit me to say it in modesty, the civil engineer ranks second to none. (Applause.) If there ever assembled a body of men as numerous as this in which there was present a greater proportion of strong men, of intelligent men, of cultivated men, of self-reliant men, of men who know what they are capable of and willing to do, it has not been my privilege to be present.

I feel proud and honored to have the opportunity to attest this fact, and, when I say it to you, I do not say it in any vainglorious spirit, but because I am impressed with the absolute truth thereof. No man can question the fact who has the privilege to look into your countenances and to address your intelligence.

We feel honored in the visit paid us by our guests from abroad in the sympathetic and enlightened spirit in which they view and inspect all our public works. I think it is impossible to manifest a deeper and a more lively interest in the enterprises of our own or any other country than that which it has been my privilege to witness on the part of these, our honored guests, in the few days which it has been my privilege to have intercourse with them. (Applause.)

Gentlemen, I thank you for the very kind attention you have given me, and submit to you the first toast, "The Institution of Civil Engineers," which will be responded to by the most distinguished naval constructor of modern times, or perhaps any times. (Applause.) In the magnitude of the work accomplished, in the responsibilities assumed, and in the stewardship rendered, he has no rival in past nor present history. (Applause.) I have the honor and pleasure to introduce to you Sir William H. White, President of the Institution of Civil Engineers of Great Britain, who will respond to the toast. (Applause and cheers.)

THE INSTITUTION OF CIVIL ENGINEERS.

SIR WILLIAM H. WHITE, K.C.B., F.R.S., PRESIDENT, INST. C. E.

Mr. President, Ladies and Gentlemen: I beg you to believe that there has never been an occasion in my life when I have felt it so great an honor to be called upon to respond to the toast which has been given—that of the Institution of Civil Engineers. The occasion is so unique, the assemblage so extraordinary, that one cannot but feel moved deeply by such kind reference and such sympathetic reception as has marked this evening. (Applause.)

Gentlemen, it has been my lot, in a life that is now advancing, although not so advanced as some of my American friends have taken it to be (Laughter) it has been my lot to be president of more engineering societies than probably any man alive. Why this should have been added to duties that were onerous in themselves, I cannot tell, but I wish, sir, if I may in passing venture to correct one statement which you made, and to claim that it is more than a century—a good deal more than a century—since there were great civil engineers. I am a past-president of a society of civil engineers founded five years before this great Republic. The Smeatonian Society of Civil Engineers still exists. I cannot say that we do much professional work, but I can assure you, as an experienced member of that institution, that we have excellent dinners (Laughter); that we have choice gatherings of kindred spirits, and that in these gatherings on many occasions we have the pleasure of receiving American engineers. And that society bears the name Smeatonian, and Smeaton was a great engineer nearly one hundred and fifty years ago, so that we go further back, and we claim more than you, Mr. President, claim for us—for you, with us, share in that past; we have a common past in engineering as well as in other branches of our history as a people, for we are not going to let you off your share of the responsibilities in all that went before that breach which happened now nearly one hundred and thirty years ago (Applause and laughter), which has seemed at times to be enlarging, but which, as a matter of fact, as the century goes on, is ever closing. You are making bridges across your rivers, and tunnels underneath to increase communication between the different parts of this great city. You on your side, we on ours, are throwing out bonds of union. The breach is closing. Cannot we say it is closed? (Great applause.)

We are one people in the past, with a common knowledge, with common aptitudes; as I said the other day in your house, common faith, common hopes, common principles; shall we not say, while holding our own and developing on our own lines, you in your great Republic and we in our great Empire, that we shall have in all essentials a common future? (Great applause.)

There hang the two flags, gentlemen, side by side. I trust the time will never come—I believe the time will never come—I am sure the time should never come (Cries of "Hear, hear," and applause)—when they won't float over the forces of two great friendly peoples, between whom there may be no formal bond of alliance, but who are bound together by ties stronger than any diplomatic documents, for one purpose (Applause), and that purpose to promote the opportunities of the world, to increase the happiness of mankind, to nourish commerce, to establish intercourse, to make the whole world better.

There is no cant in such a wish as that. (Applause.) Gentlemen, what do we want, the two great countries? We have no lust of conquest; we have no desire but the peaceful prosecution of trade and the benefit of our fellowmen, while doing a little business for ourselves. (Applause and laughter.)

We engineers deserve all that the President has said of us. Perhaps he was too modest; I think he was. Gentlemen, do you know what the word "engineer" means? I think sometimes even we engineers forget; I think even amongst engineers there is sometimes a fearful thought that it has something to do with machinery at the bottom, after all (Laughter), but although I am a past-president of the Mechanical Engineers, I scorn that idea. (Laughter.)

No, no, the true meaning is what our former Honorary Secretary of the Institution of Civil Engineers, Dr. Colgate, gave to it. It goes back to the old French verb, to devote oneself, to get at the bottom of things, to do things. That is our mission here. To quote from our old charter, it is our business to bring the forces of Nature into the service of Mankind. That is what we are always doing in many different ways, but that is our mission, and can anything be nobler? Can anything be nobler than to further the comfort and all that makes for the happiness of our fellowmen? Can anything be better than to make peoples know one another, to facilitate intercourse, to promote interchange, and so to make the whole world one —and that is the mission of the engineer.

And one need not go so far afield as that, for here in this great city it seems to me the lesson stares one in the face on every side. This city, in its situation and its surroundings, has natural advantages which, in some respects, are unsurpassed, but what would they all have been without the labors of the engineer? What use could be made of them if there had not been men who were able to deal with the situation and to turn it to practical account?

And now, as by your favor, we, your visitors, your guests, have been taken from day to day from one place to another in this marvelous region, this region of enlightenment from the engineer's point of view, we find on every side not merely monuments of past work, but we find great works in progress—progress which is so rapid as to be almost startling. We find bridges, tunnels, all kinds of transit, including that great work which we had the happiness of visiting three days ago (time passes so quickly that one almost forgets), and I hope, gentlemen, that when the Rapid Transit Subway, which is now so near opening, is opened, it will always be remembered that we of the Civil Engineers from Great Britain were amongst the first to be taken over its lines, and perhaps of all the millions who in the future will be carried, there will be none who will be more stricken with admiration, and I doubt if there will be many more competent to appreciate the magnitude of that great scheme. (Cries of "Hear, hear," and ap plause.)

And then one cannot forget these towering structures that lift their heads to heaven because the earth below is so precious. (Laughter.) Well, now, where would they be without the civil engineer? When one sees them in their finished form, in what may be called an architectural disguise (Laughter), one criticises the taste and ability of the architect, but beyond that lies a skeleton. (Laughter.) I do not know what the future of New York will be, but we have got a proverb about skeletons in the cupboard, but here the skeleton is far, far more extensive than can be put into any cupboard, in fact, it makes the thing itself. It is only another proof, Mr. President, of your general proposition that the most modest men on earth are the civil engineers (Laughter and applause), and that almost leads me to say that I must sit down at once, but there is one thing more I want to say.

Professors of pure science are now busy with one more speculation upon the constitution of matter. It is a great thing to be optimistic; it is a splendid thing to believe that what you are saying at the time is the last word on the subject, because it gives you confidence and force. It is quite true that you may have to amend it next year, but meantime be confident, at least be confident; and so we are told that now there is good evidence that electricity—whatever that may be—is the substance upon which all matter is based, of which all matter consists. Well, we have had the ether with us for many years; it is a convenient term; it enables us to imagine—to exercise the scientific imagination—and so to frame theories by which certain phenomena may be reconciled or explained, and so I have no quarrel with my brothers in the scientific world because, gentlemen, I for one venture to say that the engineer is as truly a man of science as the man who devotes his efforts to pure research. We are brothers in science, and when any one is disposed to tell me that the man who turns his scientific knowledge to practical account falls into some inferior position, I say, nonsense, nonsense. (Applause.)

There is no other way of dealing with the subject, nor is there any force in the contention that there is something mean and sordid in one's getting his livelihood by doing honest work, even as an engineer (Laughter); but coming from these speculations, we on our side have quite as high a respect for electricity as the purely scientific theorists, only to us electricity is a servant to be yoked and moulded and adapted to the needs of the universe and of mankind, and here in this great city one seems to live in an electric atmosphere; the hum of the dynamo is over it all (Laughter), and my first nights here were troubled by strange and unusual noises, and my windows were lit up by fitful flashes, and I have come to the conclusion that one reason why we Englishmen have enjoyed this visit so much is that we have been in a magnetic field all the time. (Applause and laughter.)

At all events, this I am sure of, that the magnetism of your kindness and the courtesy of your reception, your constant care for our comfort, the generous hospitality which we have received on all sides, must tend to bring about closer union between engineers here and at what we can all call "home," for though the President alluded to visitors from abroad, I claim that we are visitors from home—home. (Great applause.) And when you come there, you will come home. (Applause.)

It is true it is the old home; you have your own homes. That is all right. We have our children; they make their homes; but, gentlemen, they always say, they will go "home" so long as the father and the mother can be found, and, gentlemen, they often go home even when the father and mother are no longer there.

I heard a story in New York—and that is the last word I will say—it was only a few days ago I was talking with a workingwoman here, and she said to me, "You are an Englishman." "Yes," I said, "I am." She said, "My father was an Englishman; he came here and brought me when I was only six months old, and he is now an old man and he is at home in England." I said, "He has gone to England?" "Yes," she said, "He told us that he could not die happy until he had gone to see his home again. He was only a workingman, and his children said to him, 'Isn't it too arduous a journey, too long a journey, too difficult a journey for an old man like you?' He said, 'No, I must go home.'" (Applause.)

Gentlemen, that word is English, peculiarly English. There is no other language that has that word, and that is common to us all. (Great applause and cheers.)

The President.—Gentlemen, the next toast is "The Louisiana Purchase Exposition." It will be responded to by a past-president of of the American Society of Civil Engineers, whose home is in St. Louis, whose connection and identification with all of her most important enterprises, municipal and commercial, has been so close as to qualify him in an eminent degree to respond to this toast. I have now the honor to introduce to you Mr. Robert Moore. (Applause.)

THE LOUISIANA PURCHASE EXPOSITION.

Robert Moore, Past-President, Am. Soc. C. E.

Mr. President and Gentlemen of the Society of Civil Engineers, you, Mr. President, and Gentlemen, Members of the Institution of Civil Engineers, in addition to the reasons which have been given to you by our President or which will be given to you by those who follow me for welcoming you to our shores, we who live beyond the Mississippi have reasons that are peculiarly our own. For we, in that region, are this year celebrating our first great territorial holiday, in commemoration of the Centennial Anniversary of that most auspicious year in which the flag of Spain and the flag of France were lowered, and the flag of the English-bred and English-speaking people of the United States of America was hoisted in their place. (Applause.)

The territory which was thus added to our national domain was the whole western water-shed of the Mississippi River, a territory which, in extent, is more than equal to the area of Great Britain, Germany, France, Italy, Spain and Portugal combined.

To this great territory, therefore, gentlemen, we bid you a most hearty welcome, and we trust that you may traverse it from end to end, that you may see what marvelous results United States American English-speaking energy and enterprise have wrought for us. For, in those years, the wigwam of the savage has been replaced by the home of the white man; the prairies have been dotted over with school houses; in the wilderness great cities have sprung up as by magic, and the whole area has been covered with a network of railways whereby widely separated States have been bound together into one people—a people as alert, as resourceful, as self-reliant—and, may I say it, as open-minded and as open-hearted as any people upon whom the sun has ever shone. (Applause.) And, not content with this, our hardy pioneers have crossed the crest of the water-shed of the Mississippi River by which we were bounded, and have pressed on westward to the last limits of the land, and now we, their successors, through an American cable, touching on its way only on American soil, send greeting to our English brethren across the ocean at Hong Kong. (Applause.)

Whatever may be said of the motive of Napoleon in making this transfer—and I think little that is good can be said of him—there is little doubt that of all the acts of his life this was the one which has contributed most to the welfare of mankind. It removed peaceably a barrier in the development of a fresh and vigorous people; it placed the tools of civilization in the hands of those who have known well how to use them.

As a means of commemorating and celebrating this great event in our history and in the world's history, it has seemed to us that there was no more fitting precedent that we could follow than one which has been set for us by you in the mother country in the first great international exhibition of 1851. The government and the people of the United States, therefore, have joined with the chief city of the Louisiana Purchase, and the chief city of the Mississippi Valley, in inviting the nations of the world to join with us in an international exhibition in which each nation should bring that which will best illustrate the progress which it has made in the arts of civilization; and to this call the response has been so general and so cordial as to exceed our most sanguine expectations, and the result has been an exhibition which is certainly unequalled in extent and unsurpassed in quality.

In some respects, indeed, it is quite unique. Never before have the nations of the Orient, China, Japan and our own new possession, the Philippines, been so fully represented; never have there been gathered together so many examples of the early types of mankind. One may therefore see, not only the civilization of the present day represented in its works, but he may also see in living examples the steps by which, through centuries and millenniums of the past, this civilization has been attained.

Of the value of such an exhibition, as an educational influence, as a means of better understanding between nations, and as a means of stimulating them to greater efforts on the part of each in the future, too much cannot be said. But to no class of men does such an exhibition have greater interest than to the engineers, using the word "engineering" in the broad sense of the words of Tredgold to which you have alluded and which has been adopted as a part of the charter of the Institution which you represent, that is, as the "Art of directing the great sources of Power in Nature for the use and convenience of Man." In this sense I say that this exhibition and all such exhibitions are largely an exhibit of the work of the engineer. Everywhere may be seen the work of Watt, and Smeaton, and Telford, and Stevenson, and their thousands of successors. In fact, were it not for the steam engine, for the railway and for the thousand tools and trades which these elementary devices and inventions have made possible, the very formation of such an exhibit would have been an impossibility, if, indeed, it had not been beyond the possibility of man to conceive.

To this great territory, then, the acquisition of which by the United States marks one of the great steps of the English-speaking people in the leadership of mankind, and to this great exhibition which registers the progress which man has made to-day at the opening of the Twentieth Century in subduing the forces of Nature to the service of Man, we bid you a most hearty welcome, and may your visit add as much to your pleasure and your profit as it will add to our pride and to the prestige of our holiday. (Great applause.)

THE PRESIDENT.—Gentlemen, the next toast in order is "The International Engineering Congress," which will be responded to by a member of our Society to whom, in his enthusiasm in the profession of engineering, I cannot do reasonable justice. I will permit you to judge, yourselves, when he dilates to you upon this subject. I have the honor now to introduce to you Mr. Henry S. Haines. (Applause.)

THE INTERNATIONAL ENGINEERING CONGRESS.

HENRY S. HAINES, M. AM. SOC. C. E.

Mr. President and Gentlemen, in which expression I have the honor of including the ladies, you have all listened with interest to the remarks made by our honored Past-President with reference to the Louisiana Purchase Exposition. Now, may I be permitted to divert the current of your thoughts for a few moments to a side-show of that exposition. (Laughter.) In the words of the immortal Artemus Ward, who said that his show was a moral show, I want to say that this show about which I am to speak to you also has a moral, and before I get through with what I have to say I hope that you will see what the moral of it is.

Those of us who have not yet had the opportunity to visit the St. Louis Exposition may take it for granted that there are exceedingly interesting exhibits of the material resources, the industrial products and the works of art of many other countries, but that the only real, adequate and representative display will be from our own land, and it will be that part of the exposition which will be of most interest to our friends from abroad. (Applause.)

Now, as we Americans, or I will put it a little more modestly, as we United Staters pass along in review of these miles of our exhibits and our hearts swell with pride at the impression which we know that those exhibits will make upon the minds of these visitors as to our greatness and our power, is it not worth our while to look back to the origin of all this greatness, and to consider how much of it is due to ourselves and how much to a benign Providence.

We have inherited a splendid patrimony, it is true, but have we made the best use of it? Those who have never been abroad may think that we have; those of us who have had the opportunity to see what the rest of the world is doing may be rather doubtful of it.

It is this question which comes home with peculiar force to every member of the engineering profession in this country, for this wonderful display of exhibits would be meaningless and without intelligible connection with its purposes unless it could be associated with some rational interpretation of the processes by which all these things were brought into existence.

In the growing complexity of industrial instrumentalities, there is scarcely one of these exhibits but which owes either its form or its function to some phase of engineering activity. It is this aspect of the St. Louis Exposition which makes it a most favorable opportunity for us to compare our own engineering processes with those which have been developed elsewhere under different conditions, and more deliberately. Have we made a better use of our national resources than others have? Have we applied them to the welfare of our people with greater efficiency and economy?

It is in response to this attitude of your own minds that you, the members of the engineering profession in this country, may derive the most profit from an examination of the display made at this St. Louis Exposition.

Gentlemen, with this in mind, the Board of Direction of the American Society of Civil Engineers has undertaken the organization of an International Engineering Congress in connection with this Exposition. In doing so, the Board recognizes the responsibilities which have attached to this society for over half a century in the legitimate exercise of its function as the representative organization of the engineering profession in general in this country. Its high standard of requirements, both personal and professional, attest to the character and the attainments of the thirty-two hundred of its members, and we may even claim for it that it is a sort of an International Association in itself, since nearly ten per cent. of its members reside abroad.

The American Society of Civil Engineers would, therefore, seem eminently fitted to undertake an organization of such a Congress as being the central source from which have radiated the several specimens that have won their way to prominence since its foundation, as factors in the ever-widening field for the application of the forces of Nature to the use of Mankind.

Keeping this purpose in view, the Committee on Organization has selected from this general field of engineering certain topics for discussion at this Congress as being of immediate and increasing importance, not only to the engineering profession, but to civilization as well, for civilization is advancing with such rapidity that its progress can no longer be followed step by step for a century, in an exposition of its achievement; only for a decade at most, and any effort to set forth its processes cannot be successful that undertakes to do more. The Committee, therefore, hopes that the proceedings of this Congress may set forth the progress that has been made in the past ten years, in the improvement of processes and appliances in the several branches of engineering, rather than to be devoted to historic and biographical reminiscences and to the description of monumental work, and that the deliberations of this Congress may be the means of enlightenment to the members of this Society, to whose liberality we are indebted for the means to bring this Congress about.

In accordance with these views, the Committee has endeavored to prepare a suitable basis for the succinct discussion of these several topics which have been selected, by procuring from engineers of recognized eminence in their respective countries leading papers for this very purpose, to confine, as far as possible, the discussion to this particular phase of engineering progress, and from the opportunities afforded me to form an opinion, I believe that from the responses which have been made to the requests of the Committee, that the proceedings of the Congress—in which will be, of course, included the discussions at the sessions—will be of interest and of value as an exhibit of the most recent practice in the several branches of engineering, not only here, but also abroad.

The syllabus of subjects includes topics relating to military and naval engineering, in which the chiefs of the department bureaus at Washington have most kindly taken an interest. In our country the military engineers have the improvement of harbors and waterways directly under their charge, and you will see from their contribution to the transactions of the Congress that their ability and experience is of as much value to their country in the development of its resources as in the defense of its frontiers. (Applause.)

There is another feature of this Congress which it is greatly desirable to have properly emphasized, and that is the topic of engineering education. No one should undervalue the lessons of empirical experience, of knowing how to do things for themselves, the very basis of engineering as an art; but as the field of engineering broadens from the structural use of materials to include the application of natural forces, as engineering approaches the condition of applied science, it becomes of as much importance to know what can be done as to know how to do it, and this is the service which the engineer, in the study and the laboratory, renders to the engineer in the field.

Now, as the Committee has set apart a section for the discussion of what may be termed engineering pedagogy, it is to be hoped that that section may be made of conspicuous value by the concurrent aid of all professors of technology, and of those engineers from abroad who are here to-night I ask on behalf of the Committee that they will take an active interest in the sessions of the Congress, for, gentlemen, whatever may be the differences of opinion among us here at home as to the discouragement of the importation of foreign commodities, I believe that none of us desire a prohibitive tariff on international exchange of ideas. (Applause.)

Now that I am addressing my remarks to our friends who have been with us for this past week, I would like to conclude what I have to say by something that I have been reminded of in the very eloquent remarks of the distinguished gentleman who represents our foreign visitors here, or, I ought to say, our brothers from the old home, (Applause) by relating the experience of a friend of mine, which I do not mind telling you before my American brethren.

Well, my friend was at Saratoga Springs, which is one of our noted national watering places, and he was taking a drive around the environment, and his driver who was telling him about all the things there were to be seen, about all the fine residences and the beautiful grounds and the views on the lakes, and what great sport they were having at the races and all this kind of thing, and it occurred to my friend to ask him to drive him to the battlefield of Saratoga. Well, the man pondered for a moment, and he said, "The battle of Saratoga?" "Why," he says, "I never heard of any such battle." (Laughter.) "Oh, yes," my friend said, "Oh, yes, it was a great battle where we fought the British." "Well," the man says, "may be so, it might have been such a battle, but I never heard of it, but if there ever was such a battle I reckon it is obsolete now." (Applause and laughter.)

Now, gentlemen, I must quote Artemus Ward to you, "The force of this remark lies in the application thereof." (Great applause.)

THE PRESIDENT.—Gentlemen, the next toast in order is "The City of New York." This was to have been responded to by the Hon. George B. McClellan, who, by official duties, is prevented from being present, and therefore you may assume that this great metropolis extends to you the heartiest welcome and its unlimited hospitality; bids you Godspeed in your work, and encourages you in all your enterprises, whether National, State, or International.

We will therefore pass the toast, and proceed to the next, which is "The Civilization Engineer," a subject upon which you will be addressed by a man who has made this subject one of exhaustive study and is prepared to give you some of its most recent and encouraging directions. I have the honor to introduce to you Mr. James P. Munroe. (Applause.)

THE CIVILIZATION ENGINEER.

JAMES P. MUNROE, ESQ.

Mr. President, my British fathers and my American brothers: I hasten to repudiate the idea that engineers need civilizing. The distinguished President of your British Institute, in his very charming address, told us rather complacently of the Smeatonian Institution which, he says, was old, gave fine dinners, and did nothing. I, too, coming from Boston, would like to brag a little. (Laughter.) We have in Boston an institution that is old, that gives fine dinners and does nothing. (Laughter.) Need I say, for you Britishers are well acquainted with it, that I mean the Ancient and Honorable Artillery Association. (Great applause.) And speaking, as the last gentleman did, of obsolete battles, may I venture to say that I was born in Lexington, and my great grandfather lined the men who first defied you.

And in this connection they tell a good story of that most charming and beautiful old Englishman who is now a resident, and has been for many years a resident of America, the Rev. Robert Collier. Some years ago he took his little grandson, who is a fine American patriot, to Lexington to show him the battlefield there, and the Rev.

Robert Collier said to the boy, after explaining the details. "Well, Sonny, it was not much of a battle after all." "Well, Grandpa, anyway we licked you." (Laughter.)

I wish to thank you, Mr. President, and Gentlemen of the American Society, for the extraordinary privileges of being here this evening, and of joining in the expressions of pleasure at seeing here so many eminent engineers from the other side of the Atlantic, that Atlantic which by engineering skill has been reduced to the mere width of an ordinary pond, separating us, as Sir William White has said, from our old home. As an unprofessional man, I feel an exaltation at being in this high company; as a Bostonian I feel grateful that this huge metropolis is willing even to acknowledge that there is any other city in America (Laughter), and as a layman I rejoice in the opportunity of saying and trying to prove what I have long maintained, that you civil engineers are, in fact, civilization engineers; that you are not merely builders of roads and bridges and mills and machines, but that you are, in the supremest way, builders of a better material, intellectual and spiritual life. As an outsider, I may claim for you what you are rather ready to claim for yourselves, the first place among the forces which make for the material and moral advancement of mankind.

In your journeyings here, you have seen or will see, doubtless, many of our public monuments and statues, and with some notable exceptions, you will doubtless wonder, as we do when we look at them, why it is that individuals and communities are willing to spend such large sums of money in advertising publicly their ignorance of art. (Applause and laughter.) But with these notable exceptions, you will agree with us that the striking and important monuments here are not the work of the sculptor, but that those monuments of the railroads and the bridges, the subways and the towering skyscrapers, and by the way, speaking of those reaching heaven, we Bostonians believe that that is the only way in which New Yorkers ever will reach heaven. (Laughter and applause.) These monuments, we believe, and we think you will agree with us, are the great monuments of this country. Those statues of marble and of bronze are static; they represent past events which our later experience may have repudiated, but these monuments of the engineer are glorious, dynamic evidence of the splendid future that is to be.

Once upon a time, we Bostonians permitted a leaden statue of King George III; later we decided to melt it into bullets to fire at His Majesty's troops.

Not many years ago, in this City of New York, they erected a triumphal arch, which, I believe, is no longer standing. (Laughter.) In the Boston Athenæum there stands a row of marble busts unlabeled, for the reason that, although fifty years ago those men were great, to-day not one human being can identify them. (Laughter.)

But you engineers, on the other hand, are every day building your own monuments; every day, in laying the foundations of your structures, you are laying more solidly the foundations of human welfare; every day as you uplift those structures you are uplifting all mankind; every day as your workmen drive in the bolts, you engineers are bringing closer and closer together the whole human family. Every day, silently and steadily, you engineers are transforming the destructive agencies of the old military engineer into the constructive agencies of the civil—or, as I like to call him—the civilization engineer.

There is no doubt that you engineers are the supreme motive force of modern civilization, first, as I have said, because your work is dynamic and not static; secondly, because in everything that you do, more than anything any other man has to do, you have to be obedient to absolute and immutable laws; and thirdly, because in eagerly seeking out those laws and using them for the benefit of mankind you are freeing man, making him the master where he was formerly the slave of Nature (Applause), and to civilize a man is simply to make the animal side of him independent.

The hungry man can have no thought except of his insistent appetite; the isolated man can have no ideas beyond the narrow range of his horizon; the man bound down to grinding toil can never lift his mind and soul above his daily torture; but you engineers, by your work, are making the hungry man and the hungry nation to disappear; by your labors you are bringing the isolated man into commerce with all the world; by your inventions you are replacing the aching and strained muscles of the manual laborer by the tireless machine.

A pessimist is said to be a person who has to live in the house with an optimist. (Laughter.) If that be true, then my family must be sad pessimists, for I have abounding faith in humanity, and I see no immediate or future danger, as many do, in the tremendous power of wealth that the labors of you engineers have, in the last half century, created, for it is your work alone that in this last half century has created these colossal fortunes, these enormous combinations of business, this speculation and promotion on a perfectly incredible scale; but these evils appear gigantic as compared with similar evils of the last generation, simply because everything now, the whole scale is itself colossal.

There has been a tremendous advance of the whole social scheme, and, while that social advance has made rich men incredibly richer, it has also lifted up at the same time thousands and hundreds of thousands of men who, under old conditions, would have known nothing but hopelessness; it has lifted out of utter degradation thousands and tens of thousands of others who are now decent citizens because of the work of you engineers.

As I have said, I am not a professional man, but in connection with the Massachusetts Institute of Technology, which I am sorry you are not to visit as an organization, and which I hope you will visit as individuals before you go back—in connection with the Massachusetts Institute of Technology I have something to do with the training of young men, and it seems to me that the most important, as well as the most difficult, problem before all of these schools of applied science is to make the young graduate understand the tremendous power that he has in civilization, the colossal part that he is to play in the future of the world.

The average undergraduate fixes his mind upon professional details, upon what we may call the knacks and tricks of the trade. The duty of those schools should be to make those young men see that they are not to be simply engineers; that they are to be leaders in civilization. Therefore, the work of those schools is not simply to teach engineering; it is to compel those men to have a broad outlook upon life, a wide outlook upon men, a noble outlook upon the professions which they are to follow; and unless those schools and those colleges meet this, unless they train these young men in this broad way, then they will not be fit to be engineers, and they will not prove to be fit successors for such men as you. (Applause.)

The President.—Gentlemen, the next toast is "The Army." It will be responded to by a gentleman who hails from the State of Ohio, the Mother of some Presidents; bounded in part on the east by my native State, Pennsylvania, and partly on the west by my State of adoption, Kentucky. I have now the honor and pleasure to introduce to you Lieutenant-Colonel G. J. Fiebeger, of the United States Army.

THE ARMY.

Lt.-Col. G. J. Fiebeger, U. S. A., M. Am. Soc. C. E.

Mr. President, Ladies, and Members of the Institution of Civil Engineers of Great Britain, and Fellow-Members of the American Society of Civil Engineers.

The essence of warfare was aptly put by that old Confederate Cavalry raider, Gen. Foster, when he said, "I don't know much about your maneuvers or other fancy business, but I do know that the feller that gets thar first with the most men is mighty apt to win." (Laughter.)

That is the great problem which all great commanders have been trying to solve from the time of Alexander to Kuropatkin. (Cries of "Hear, hear" and applause.) And I am not certain but what the latter gentleman might add something about the necessity of staying there when you get there.

Now, it was in solving this "getting thar" problem that we of the the military profession believe we founded that great profession of engineering. We believe that it was the pioneers of the armies of Alexander, Hannibal and Cæsar who opened the roads for their advance through primeval forests, over marshes, through mountain gorges, who founded that great army of road-builders who are to-day connecting the oceans with their bonds of steel and making the maps of the continents look like wire entanglements with their intersecting railroads.

As I look over the names of those present and see the names of so many eminent members of that army of road-builders, I feel proud to belong to a profession that started you on your road to fame. (Laughter.) We also believe that it was the pioneers of those early armies that were the forerunners of the great army of bridge-builders which is to-day spanning the rivers of the whole world in the interests of commerce, just as their founders did in ancient times in the interests of war. Ever since reading in Cæsar's Commentaries the description of his bridge over the Rhine, I have had a tremendous admiration for the army of bridge-builders, and since I have had the personal acquaintance of some of the designers of those gigantic structures which now span the Niagara River, my admiration has greatly increased. I believe to-day we would be reading of projects for spanning the gap between Mars and Earth with wire or eye-bar cable (Laughter), with cantilever of steel or concrete structures with suitable architectural approaches (Laughter), were it not for the instability of the foundations and the uncertainty of the span.

I am, therefore, proud that we, of the military profession, are the sponsors for the bridge engineers. I might go further into the province of engineering, for my friend Herschell wrote as well upon the strategems of war as he did upon the waterworks of Rome, but I am afraid I would tire you. and therefore I would rather turn to the other side of the picture and show what the military profession owes to the engineers.

Modern warfare would be impossible were it not for the railroads constructed and operated by engineers. The first great commander to acknowledge that was General Sherman, who tells us in his Memoirs that his celebrated Atlantic campaign would have been absolutely impossible were it not for his admirable corps of road and bridge builders. He tells us that these gentlemen became so expert in the construction of roads, railroads and bridges, and even tunnels, that they could construct them much more rapidly than Forest and Joe Wheeler could destroy them. This was a fatal blow to those gentlemen, and also to the Confederacy.

It was due to the magnificent railways of Germany that Von Moltke, in 1870, was able to get to the frontier first with the most men, and conquer France.

In the latter part of the Nineteenth Century, when General Kitchener, the Sirdar of Egypt, decided to move (Applause) to overthrow the Mahdi at Khartoum, he remembered two comments of Napoleon. The first was that a great desert is a more formidable obstacle in front of an army than a great river or a great chain of mountains, and second, that he who proscribes the aid which the engineer's art gives to the commander in the field deprives himself of an auxiliary force and expedient always useful and sometimes absolutely necessary. The Sirdar, therefore, called to his aid the road and bridge builders who laid his tracks across the great Nubian Desert; he built his Atbara Bridge, with the assistance of the American Society of Civil Engineers, and he reached the field of hostilities, if not with the most men, at least with a few of the best, and a thin red line reconquered the Soudan. (Applause.)

These are but a few of the examples in which the fighter and the engineer have operated together in the field, but they are sufficient to show that scientific training is absolutely essential to the soldier, and that the commander of future armies, if not an engineer like McClellan, Gen. Lee and Gen. Kitchener, must at least have sufficient scientific training to appreciate, like Napoleon and Gen. Sherman, that the engineer's art in the field is always useful and sometimes essential.

The future of warfare is destined to see a great expansion of the engineer's art in the military field. Besides the road and bridge engineer, the mechanical engineer and the fortification engineer, the Army must call to its aid the electrical engineer for running the searchlights, to operate its submarine mines, and also to operate its great seacoast guns. It must call to its aid the mining engineer to run the tunnel approaches to fortified places, and thus avoid the shell which makes the advance over the surface to-day almost impossible. It must call to its aid the sanitary engineer to reduce the mortality in its great camps of concentration. The very term "scientific warfare" means warfare in which the engineering art is bound to take a prominent place. (Applause.)

The President.—Gentlemen, the next toast is "The Navy." This was to have been responded to by Francis T. Bowles, Esq. Inevitable absence devolves that important duty upon the constructor of the greatest battleship of modern times, the *Connecticut*. I have the honor to introduce to you Naval Constructor W. J. Baxter.

THE NAVY.

W. J. Baxter, Esq.

Mr. President, it is very regrettable that Admiral Bowles, the late chief constructor of our Navy, is absent; it is regrettable for your sake and for mine. Mr. Bowles knows all about our Navy, and he knows how to make a speech. I know something about the Navy, but I don't know how to make a speech. Sir William White knows about all of the navies and he knows how to make a speech. (Applause.)

The one thing that has struck me here this evening is the great modesty of all of the speakers. Now, the Navy certainly goes as far back as the time of Father Noah. He certainly was an admiral, and he may not have been the first naval constructor, but he was a naval constructor as well as an admiral. Therefore, I think that my profession is the oldest one represented here to-night. (Applause.)

The battleship of the day, gentlemen, is the combination, is the representation of the united engineering talent of the world. Every part of the engineering profession contributes to its development, its design, its building and its use.

Now, I have heard talks of engineers who were interested in tunnels. Well, you can go down in the shaft alley and you can get a very good example of a tunnel. If you are interested in bridges, why, any battleship that has not got to have four bridges is of no account at all. If you are interested in railroads, well, what are your railroads for? Simply to bring things to build battleships with. You are interested in harbors. What are they for? They are simply places in which the battleships can go. And so it goes.

Now, this electric galvanic spark, the battleship, is permeated with electricity all the way through, and Sir William White surely has forgotten some of his nights on board battleships when he thinks of his nights in New York.

There is one thing, gentlemen, that possibly some of you don't know, that to Sir William White, more than any man outside of this good country of ours, belongs the development of the new navy. The men who designed and built it in the first place were brought up under his tuition. (Applause.) Since our feathers have begun to spring, we have tried to do some designing on our own account, and we have been a little bit slow, so that where we bring out a battleship of 10 000 tons, Sir William would bring out another one of 11 000, and immediately we bring out a cruiser of 12 000 tons he would bring out one of thirteen, and so it went along.

Now we are building a class of ships, one of which I am connected with, the *Connecticut*, of 16 000 tons, which we consider the very finest battleship in the world, and when the *Connecticut* is put afloat to follow her sister ship, the *Louisiana*, which was put over two weeks ago, those ships are to-day the most powerful battleships afloat, but I won't say anything about Sir William, but somebody over there has got something on paper which is a little bit larger than the *Connecticut*. (Laughter.) Now, we won't stand for it, and we will beat her next time.

There is only one other thing. It has been said that the civil engineer is the civilization engineer, but the most powerful civilization engineering element in the world is the Navy, and the country which has the Navy is the one which controls civilization. (Applause.)

THE PRESIDENT.—Gentlemen, the next toast is "The School," which will be responded to by a Member of the American Society of Civil Engineers, the President of the Stevens Institute of Technology. I have the honor to introduce to you Alexander C. Humphreys.

THE SCHOOL.

ALEXANDER C. HUMPHREYS, M. AM. SOC. C. E., PRESIDENT OF STEVENS INSTITUTE OF TECHNOLOGY.

Mr. President and Fellow-members of the Institution and the Society.

"The School," and, before this audience, that must mean the School of Engineering, is a large subject to cover in my allotted time of ten minutes.

Certainly, then, I cannot discuss details of curriculum and methods of instruction, nor do I suppose these matters would be of interest except in a few individual cases.

But there are certain broad questions of Engineering Education and the connection between the School and Practice to which I will ask your attention.

Permit me to say that I shall speak from the experience of thirty years' practice as an engineer, followed by two very strenuous years devoted both to practice and technical education; and from the experience gained from supervising the post-graduate training of the members of my several Cadet-Engineer Corps. And let me add for the benefit of our visitors, that these young graduates of Engineering Colleges, while thus obtaining post-graduate training, earned, or anyway were paid, at least $50 per month, or say £120 per year.

And this brings me to my first point: The work of the Engineering Schools, even the best of them, is often misjudged through the ignorance of employers who expect their cadet-engineers at once to show practical proficiency in any and every branch of Engineering.

The partial exercise of a limited intelligence should promptly make it plain that the best four years' engineering course can only prepare the students to equip themselves promptly and surely to be efficient workers in any one engineering or industrial specialty.

Graduates and employers alike should appreciate that a graduate degree in engineering, while undoubtedly well earned as compared with other graduate degrees, does not indicate that the holder is qualified for immediate successful practice in Commercial Engineering.

The man is yet to be proved as well as the education. And even if the individual's manhood proves sufficient, there is much yet to be learned in the School of Experience.

Nothing can take the place of experience-learning, whether in Engineering or any other line of human endeavor. Certain rare personalities have been able to learn so much in the School of Experience without the advantages of previous technical education, that some who take the exception as the rule have made comparisons unfavorable to the so-called theoretical engineer and favorable to the so-called practical engineer.

But it is no longer a question between these two systems of education, for if we except geniuses, the man who to-day wishes to gain real success as an engineer must avail himself fully of both systems.

I appreciate this is all trite, but because these truths are not kept in mind the value of technical education is misjudged even within our own guild.

To go a little farther, the engineer of to-day not only must be practical as well as theoretical and theoretical as well as practical in the field of Engineering, but he must be practical in the field of business.

We admit at once that the engineer should constantly have in mind the obtaining of an adequate return on investment in connection with his engineering designs and enterprises. Then it follows that the engineer-student should have some instruction along these lines, including as a foundation the principles of accounting, depreciation, shop-cost, analysis of data, law of contracts and business methods in general.

At least the student should be convinced before graduation that to succeed really in his profession he must practice within commercial limitations.

In this connection one good sign of the times is that a number of our best engineering colleges now call in practical, commercially successful engineers to lecture on engineering practice.

But, first of all, we must be sure that our students are thoroughly grounded in the fundamentals of science and mathematics, and the constant pressure to extend the curriculum should be resisted where it threatens this thoroughness in elementals.

In teaching even these elementary subjects, practical applications should be employed. Where apparatus is used the students should be required to set it up and follow as far as possible the lines of original investigation. The cut and dried element should be reduced to a minimum. We should cultivate in our students the power first to reason and then do, rather than to memorize equations and the like. The memory should be cultivated not by the effort to store a mass of dry facts, but in connection with the exercise of the reasoning powers.

Of late years there has been much unrest in regard to technical education. Constantly it has been the object of investigation and criticism. This has been largely the result of the late marvelous progress in Engineering Science and the effort to reflect this progress in the work of the technical schools.

Much good has resulted and some harm.

As new applications of Engineering Science have developed, the effort has been made to introduce them into our engineering courses, until these new applications threaten to crowd out the very fundamentals.

Too often, even in the case of capable students, there is not sufficient time left for recreation, meals and sleep—that is, for the maintenance of full physical vigor.

And this crowding of the curriculum and the introduction of educational fads is to be found in our preparatory schools and has already resulted in superficiality.

In preparatory schools and technical colleges the students are taxed beyond their powers until their minds are numbed.

Certainly, then, no farther additions can be made unless balanced by equivalent eliminations.

It has frequently been proposed to correct this trouble in the technical schools by adding a fifth year.

But this could be only a temporary remedy, for new applications of Engineering Science will continue indefinitely to be developed.

Nor can our entrance requirements be materially raised in view of the fact that the preparatory school students are already strained to the limit.

Let us then acknowledge that the race is an unequal one and that there are limits to our students' power of endurance.

Unquestionably, it would be of great advantage if we could give our undergraduates training in all practical applications of Engineering Science, but, as this cannot be, we must console ourselves by reflecting that after graduation they will settle into specialties and perfect themselves therein in the school of experience, supplemented in some cases by post-graduate work in the university. We can still further console ourselves by investigating the records of our Alumni.

Stevens Institute, for instance, furnishes its students only a single prescribed course, with no electives. Its secondary title is "A School of Mechanical Engineering," which seems to indicate a rather narrow specialization.

Still, the Alumni Directory shows that its men are occupying nearly every class of position in the field of Engineering.

Three notable examples taken from the list of Alumni will serve to show the wide range of professional work performed on the original, fundamental training furnished by the Stevens curriculum of twenty years ago.

One of these, an eminent authority on Dock Engineering, has been active in the entertainment of our guests; another is this year's President of the American Institution of Electrical Engineers; the third is this year's President of the American Gas Light Association.

Let us recognize that the days of the Greek and Roman civilization, when the educated man was expected to absorb the whole store of knowledge, have passed. Now we must be satisfied to know a little of many subjects and be a master of one.

Nor need we engineers, if we follow this line of thought, fear to be criticised for being narrow specialists. This condition applies to all the world's workers.

Those who so criticize us may know a little of many things but probably they are master of none.

But when we claim that thoroughness in a particular line does not necessarily imply narrowness, we must be sure that our engineer-students are held to as strict an accountability with regard to their general studies as to their technical studies, and especially so with regard to the study of the mother tongue.

It should be recognized that on account of the heavy pressure under which they work, these students are disposed to slight the general subjects of the curriculum. Then the professors and instructors in these departments all the more should be selected because of their enthusiasm and ability to interest their classes.

While referring to the instructors, let me say that I agree with Professor Perry, of England, that all the engineering professors in a college of engineering should not only be permitted, but required, to practice their profession: and this for three reasons:

First.—That they may be better qualified to give practical instruction;

Second.—To reduce the chances of their growing stale as teachers; and

Third.—That men of higher caliber may be secured.

By better teaching we stimulate better learning.

Because our students are worked to the limit of their endurance, no effort should be spared to include in their college life everything available that may tend toward legitimate recreation and the cultivation of college spirit. To this end we should have attached to our engineering schools, Dormitories, Commons, Unions and provisions for reasonable attention to Athletics.

Besides forcing these men to an unvaried grind of technical work, we should give them the opportunity to develop, through contact with their fellows outside of the class-room, but during this period of great mental activity and receptivity, breadth and the capacity for dealing with one's fellows.

We have of late been thoroughly inspected as to our teaching methods by Mosely Commissioners and individual investigators from abroad.

I am sure these investigations have been profitable to investigators and investigated alike. But in one point at least I think nearly all the investigators fell into error, namely, in forming the opinion that we in the United States have been sufficiently supported in our efforts to advance technical education by the generosity of our rich men as a class.

I hold that this is quite opposed to the facts.

It is true that we have a few notable examples of such philanthropy, but many of the benefactions have not come from rich men and therefore have been individually small.

Stevens Institute has received practically all of its endowment from three men.

One (the founder) a pioneer engineer; the second its late President, who made his modest competency by the sweat of his brain; and the third a man who owes his vast wealth to unique success in the industrial field.

This country undoubtedly owes its prosperity in considerable measure to the work done by its technically educated men, and this I affirm while having clearly in mind the many other agencies resulting from our geographical position, political institutions and Nature's prodigal benefactions.

In this prosperity all classes have shared. Of our rich, the bankers and merchants have profited as well as those directly interested in industrial pursuits. But it is not often you will find that the merchant or banker has returned to technical education even a small percentage of that which technical education has given to him.

And the cases are almost as rare among those who have acquired their wealth directly from industrial pursuits.

The credit should be given to the few rich and the many more of moderate means who have supplied the endowments and to those who have so efficiently administered those inadequate provisions. I personally know of cases where large returns have so been obtained from relatively small endowments at the cost of long-continued, heart-breaking worry and final collapse of those in active control.

When in studying our own needs and possibilities and then looking afield to our sister institutions, I see how much more could be done for the struggling young men of this country and for the relief from crushing cares and responsibilities of the workers in this field if the rich men of the country would as a class recognize their debt to technical education, I for one do not propose that credit shall be given where it is not due without raising my voice in protest.

In conclusion, let me say that in spite of their faults and weaknesses, some preventable and some not so as yet, one thing can be claimed for our technical schools:

They are doing a grand work for the country and mankind.

THE PRESIDENT.—Gentlemen, the next and the last toast is "The Law." Every engineer knows that constructive emergencies and situations have laws of their own which cannot be disobeyed if success is to follow their work. The speaker will differentiate to you between these several laws. The engineer is a law-abiding citizen, and takes his instructions from that great science, an expounder of which will now address you, the Honorable William W. Porter, I have the honor to introduce to you.

THE LAW.

HON. WILLIAM W. PORTER.

Mr. President, and Gentlemen, Members of a sister profession. The hour is late, I bid you good morning. I am under contract to speak for ten minutes, and no more, and I am just within a few moments offered a bonus to cut it down by the minute.

I do not wish to sail here under any false pretences. It has got abroad that I am a sitting judge. It is true that I did sit on the Appellate Court in Pennsylvania for a number of years, and there acquired the habit of silence. I resigned from that and still cultivated the habit. It was before the Democratic Convention, but it had no relation thereto. (Applause and laughter.)

Gentlemen, after I received the invitation in writing, with certain limitations as to time which I have alluded to, I found I was up against a proposition of a very serious magnitude, and the first thing that impressed me was my colossal ignorance of everything relating to engineering. So bad was it that I went to a Standard Dictionary to find out what engineering is.

Now, I speak seriously when I say that I have listened to-night and heard from the lips of one of your most distinguished guests one of the best after-dinner speeches I have heard in years. (Applause.) I have enjoyed that with you, but I have had a comfort in it that you didn't have. He *facile princeps* was compelled to define an engineer. (Laughter.)

What is engineering? Listen to the dictionary. (Laughter.) This is it, *in hæc verba:* The putting through or managing skillfully or by contrivance and effort. (Laughter.) And after all this laudation of gentlemen engaged in doing something by contrivance and effort. (Laughter.) I know of no collegiate degrees founded on those claims, but see here, the nomenclature of this science is getting very complex; the names of the engineers are by no means clear as indicating their purpose in their profession. Here I was led away from the dictionary and went to a more pleasing source. I took a lay woman and asked her what she thought the following designation of engineers meant, and the first thing I asked her was, What do you think a mechanical engineer is? "Oh," she said, "an automaton."

(Laughter.) Then I asked her, Well, what is an electrical engineer? "Well," she said, "that may be a storage battery or a dynamo." (Laughter.) Then I asked her, What is a marine engineer? And she thought for a moment. "Well," she says, "that is descriptive of the most innocent and ignorant class (Laughter), because from time immemorial, when we are telling something beyond credence, we refer to telling it to the marines." (Laughter.) Then I asked her, What is a sanitary engineer? "Well," she said, "I think that must be a healthy sort of person, perhaps by his own presence having some hygienic influence." And then I went back to my dictionary to find out what a civil engineer is.

Now, you cannot find it defined as it should be, so I went to the adjective "civil." What does that mean? It means a person who is pre-eminently careful and observant of all the amenities of social life. (Laughter.) Coupling that with the word "engineer," you have your description, and as to the civility of every one, the guests to-night, as well as I hope the hosts participant, have had it proven on this occasion, but the list is not complete. Why, we have got more engineers than that. We have got in America, we have got what is known as a political engineer. (Laughter.) That is synonymous, perhaps, with "boss." More than that, we have financial engineers, and the lady to whom I spoke asked whether that was not included under the term "hydraulic engineer" because of the water. (Laughter.)

But when my dictionary had sufficed so far, I went a little further by some accident, and I found another definition for an engineer. "A plotter," and a plotter with oblique morality, and it is a fact, Shakespeare himself seems to have a very low opinion of you, because, as you will remember, in Hamlet, after the killing of Polonius, Hamlet makes certain arrangements with certain persons of evil design, and finally ends by saying, "'Tis the sport to have the engineer hoist by his own petard."

Well, now, gentlemen, if you will bear with me, I want to make my bonus if I can—if you will bear with me I want to say that, having now demonstrated my absolute ignorance of everything connected with your profession, and having diplomatically put myself, by adverse criticism, wholly out of touch with everybody, I have moved by these stages up to the topic which has been assigned to me, namely, The Bar. (Laughter.) I had a serious purpose, gentlemen, when I came here, but the hour and the occasion and the limitations of contract lead me to observe that "A little folly now and then is relished by the wisest men."

Now, the Bar. Well, if I could not make you fatigued by just a little essay on the Bar, with a few extracts from Coke or Littleton, and perhaps drawing something from our own judges, even the sitting as well as the dead, I think you could sleep sounder, but I propose to avoid that proposition and just to touch for a minute or two on one or two curious episodes or incidents which have come to my attention recently, which go rather to the lighter side and relieve that stress which all judges and all lawyers are under.

There was a gentleman of the Bar, (I will not say of which County or State) who prided himself on being a great cross-examiner. One of the greatest English judges said in respect to cross-examining, "When you want to cross-examine, don't." This gentleman had upon the stand a yokel, an ignorant lad or young man, and had pinned him down so that he had got an admission which he wanted, and then asked this question: "And so, after all, you are nothing but an idle, good-for-nothing vagabond, isn't that so?" To which there was a good deal of silence. Then he took another tack. "Well, what does your father do?" "Well, he doesn't do nothing either." "Then he is just the same idle vagabond that you are, is he?" "Well," he said, "you can ask him; he is in the jury box." (Laughter.)

There was another funny slip of a word by a young lawyer who was very earnest and was trying, on the criminal side, a case of bigamy. He had earnestly urged the acquittal of his client, but in order to make the status of the law particularly clear to the jury, he said, "Gentlemen of the Jury,—" and he was in some haste, more haste than I can articulate in—"Gentlemen of the Jury, a man can get greatly complicated in his marriage relations. Under the law of this country, there are three ways that a man can be married. There is such a thing as polygamy, and that does not keep a man out of the United States Congress; (Laughter) but there is such a thing as polygamy, and that is when a man has got a plurality of wives. And then when a man has got two wives, that is bigamy; but when a man has only got one wife, that is monotony." (Laughter and applause.)

It is sometimes said, you know, that lawyers, by their education perhaps, or by the inheritance from England, have a circuitous way of answering any direct question of a client, and perhaps it is true. I think one of the best illustrations of an indirect answer which is peculiarly clear was given in a little rhyme which went current about a year or two ago, which you will remember. It recounts the proposal of a young man and his answer from the maiden. It runs thus:

"A young man asked a maid to wed.
"Go ask Papa," the maiden said.
The young man knew her Pa was dead;
He also knew the life he'd led;
He understood her when she said
'Go ask Papa'." (Great laughter and applause.)

Now, gentlemen, I really feel that you have paid me a great compliment in listening to me without being wearied, and I want you to bear with me one minute more, and I will not trespass on your want of intellect further; I want a serious word, if I may, before I sit down.

Sometime ago I had the honor in my native city —or town—I say that in New York, town of Philadelphia. You observe that, differing from my Boston friend, that is the first time I have told you where I came from. (Laughter and great applause.) I want to add in this connection that there is not a bust of a big man that has lived in Philadelphia whose bust and name is not known and honored. (Applause.)

One word seriously. Sometime ago in the town which I have named, I had the honor of participating in a welcome to that great English jurist, statesman and diplomat, Lord Herschell, and I had the pleasure on that occasion of suggesting the thought I conclude with.

No man can be otherwise than blind who has not seen cropping up, wherever Englishmen and Americans have met, officially or socially, a growing tendency, a warming up, a fraternizing in the last few years, which, perhaps, did not exist before, and we have been thinking, on both sides of the sea, why that may be. It has been suggested that the common language is drawing us. I believe it is not so. It has been suggested that the common blood draws us. I believe that that is not the reason, for Americans, if they are anything, are composite, with a large preponderance, perhaps, of the Anglo-Saxon. It is said that the literature, being in common, has drawn us together. I think that is not the real cause. Others have said that commercialism is binding us closer, and I say, God forbid; let it be on a higher level. There is one thing that I believe lies at the root of this fraternalism which shall have no evanescent being, but which shall last and grow, and it is found in this, that at home— may it be, shall I say, as others have said, or here in the new home, the same fundamental thought of right and justice exists. (Cries of "Hear, hear" and applause.)

The same methods of judicial determination are existing in both countries. No man there or here can be brought before any court of administration; no man there or here can be tried save by a jury of his peers; no man there or here can be stripped of liberty, life or property, without due process of law, under the eye of a trusted chancellor or judge. (Applause.)

When I say that to you, gentlemen, I must set you back of the position of the Bar and Bench; I must set back the profession of war, whether on the water or the land, and put first the profession of the law, which secures to every man, in every profession, in every business, equitable determination and protection of his rights. (Great applause.)

The President.—Gentlemen, your patience for a few moments longer. We have had the pleasure of hearing from Boston, from New York and from Philadelphia. We have yet to hear from the Windy City of Chicago. I will have the pleasure to call upon Mr. Robert W. Hunt to tell us of that city. (Applause.)

ROBERT W. HUNT, M. AM. SOC. C. E.

Mr. President, Fellow Members of the Institution, and Fellow Members of the American Society: This is an unexpected honor—an unexpected honor for my city in this city (Laughter), and therefore, sir, you, knowing as well as you do, as you are close residents to us, how loath we are to speak of ourselves, will sympathize with me in restraining myself to almost the words of simple thanks; but in making them, it is my pleasant duty to extend the words of welcome to our foreign friends.

It will be our fortunate pleasure to show them that city. I am no prophet and I am loath to make predictions, but if we cannot give you better weather than you have had here—(Laughter) fortunately you are only going to be with us two days and, therefore, our task will not be so great, but all the tears and all the water which you will see shed there will be upon your departure. (Laughter.)

It is a great pleasure to me, Mr. President and Gentlemen, to add just these few words to this welcome which is extended to-night, because it was my great, good fortune to participate with the American Society or its representatives in the welcome which the Institution gave to us in London in 1900, and who that participated will ever forget it? (Cries of "Hear, hear.") To you Englishmen, you cannot appreciate what it was to us, but perhaps the thing of all, which we will remember the longest, was that glorious day at Windsor. There, on that classic spot, and under those magnificent oaks, and under those historic towers, to receive that gracious welcome from the noble woman, your Queen of blessed memory, is a thing that no one who participated will fail to remember as long as memory lasts to him. It was, perhaps, a greater occasion to us than it could possibly be to you Englishmen who participated with us.

We cannot offer to you welcome to the classic scenes that you gave to us and opened to us. It is all new that we ask you to look at. There it was with reverence, and memory dating back through the centuries that we paid our visit.

Now comes the question, is the new better than the old? And it is our duty to try to make it so. Whether we are performing that duty, we have to leave it to you to judge.

It is true, as has been said, that we are of a common heritage. Perhaps England and America have not always exactly agreed. I think history shows that we have had some troubles—and what family has not had trouble—and I believe it is also, and may it always be so, that let those quarrels and those troubles be as severe as they may, it is not pleasant for the outside party to interfere. (Laughter.)

We, in our own short history, have had severe family quarrels. The North and the South disagreed; it brought sorrow and it brought bloodshed; but, thank God, that has passed. Our heroes who laid down their lives for the North are to-day also the heroes of the South, and their heroes who died are to us cherished memories of men who gave up all in the defense of that which they believed to be right, and the very blood that was shed is the cement that to-day binds together our nation. (Applause.)

So, why should it not be between you and us? Why should we not be bound together? Those flags (our President alluded to them); they are hanging together. Why should they not always float together? (Cries of "Hear, hear.") Look at them! The colors are the same, arranged differently, perhaps, but no foreign tint is there. (Applause.) It is the red, white and blue. (Cries of "Hurrah!") They have always been and they always will be, sir, glorious flags under which to live and prosper, glorious flags under which to fight—aye, and if need be, to die. (Applause.)

SIR WILLIAM H. WHITE.—Gentlemen, I am not going to appeal to your patience; I am going to give you pleasure. I ask you, without any words from me, to drink to the health of the President of the American Society of Civil Engineers, and you, Englishmen, show what Englishmen can do in three cheers to the health of the President of the American Society of Civil Engineers, Mr. Hermany, our kind host and friend.

(Three cheers given.)

INTERNATIONAL ENGINEERING CONGRESS

UNDER THE AUSPICES OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS,

ST. LOUIS, MISSOURI,

OCTOBER 3d to 8th, 1904.

REPORT IN FULL OF THE OPENING AND CLOSING SESSIONS

OPENING SESSION.

The International Engineering Congress, held under the auspices of the American Society of Civil Engineers, convened October 3d, 1904, in Congress Hall, Administration Building, Louisiana Purchase Exposition, and was called to order by Mr. Henry S. Haines, Chairman of the Committee in Charge, who spoke as follows:

Somewhat more than a year ago, the American Society of Civil Engineers was invited by the Board of Direction of the Louisiana Purchase Exposition to undertake the arrangements for an International Engineering Congress. The Board of Direction of the Society appointed a Committee of its members, and this Committee invited the co-operation of the other National Engineering Societies. But for some reason satisfactory to themselves, they did not entertain the proposition favorably. But as the responsibility for the successful inauguration of this International Congress rested with the American Society of Civil Engineers, in fact, had been placed upon it, by the management of the Exposition, the Board of the Society determined that the Society would undertake it alone.

A Committee of Organization was appointed, and that Committee determined to confine the discussions of the Congress to a few live topics, rather than to endeavor to cover the whole field of engineering, and to provide for a profitable discussion of these topics, the Committee invited leading papers upon them from engineers of recognized eminence in their respective countries. There has been a very gratifying response to the invitations of the Committee. We have papers from engineers not only in our country, in North America and in South America, but from the three other Continents of Europe, Africa and Asia; from the shores of Great Britain, and from France, and from those of Japan. I think, therefore, that this Congress may well lay claim to being an International Congress, and we may hope that the oral discussions in its several Sections will be of great interest as indicating the experience and the most recent practice of the distinguished engineers who are here present with us, including a delegation from the Institution of Civil Engineers, with its honored President, Sir William White.

As Chairman of the Committee on organization, I have now the honor to introduce as the President of the International Engineering Congress, Mr. Charles Hermany, the President of the American Society of Civil Engineers.

Mr. Charles Hermany.—Gentlemen, engineers of all nationalities, speaking many different languages, from all quarters of the globe, hailing from countries whose histories are chronicled in the lapse of centuries, Slav, Teuton, Mongolian, Gaul, Latin and Anglo-Saxon, I greet you all as brothers. You have come to us and have assembled upon a spot where an epoch in our national history was made, where to this day there has been but one hundred years of civilized government; in a territory the acquisition of which by peaceful diplomacy, that made our then infant republic extend its dominion from the mouth of the Kennebec River in the East to that of the Columbia in the far West, from the Atlantic to the Pacific. The Louisiana Purchase, in the annexation of which our first move toward territorial expansion, our first step in the direction of a world power, was made; a territory whose brief one hundred years of eventful history fills us with pride, since it has proved the key which unlocked the portal and swung back upon its hinges the western gate that has admitted our star peacefully into the role of empire; this place, from which two of our pioneer engineers, Captains Lewis and Clarke, commenced, conducted and completed an exploring expedition to and from the Pacific Ocean at the mouth of the Columbia River, that was the most remarkable in the history of the world; here, of all places in our country most suitable, we have chosen to hold an International Engineering Congress, the peer of which the future alone can produce, and to which it is my privilege and honor to bid you a hearty welcome.

We meet in a city improvised as if by the wave of a magician's wand, for the display, examination and study of the fruits of agriculture, commerce, manufacturers, art and science from all the world; where we feel the poet's "One touch of Nature" that "makes the whole world kin," and the engineers thereof brothers; under environments that stimulate us in our deliberations, and enable us to link the past, present and future into a chain that binds ages and races of man into a common brotherhood, and their civilizations into a common story or into a drama of many *personæ*. Here we are in touch with the best of research and conclusion, of thought and action, of word and deed, of endeavor and accomplishment, and of hope and fruition, that the engineers of the world have to offer. We are here assembled to enjoy the heritage bequeathed by the past ages and to prepare the way for those to come; to see, weigh and measure each other by the varying standards of our respective selections or contrivance; and to see and esteem ourselves as others see and esteem us; in fine, we are here to move the world, not with a lever from the fulcrum sighed for by Archimedes, but by the consensus of modern thought and the combined use of all the material agencies through which we strive for the ultimate complete dominion of mind over matter.

At the time of the Louisiana Purchase, this place where we are assembled was a wilderness, more than a thousand miles west of our center of population. It is now but a few hundred miles west thereof, and will, ere two decades shall have elapsed, claim, not only the center of population, but also the center of wealth.

You thus perceive that we have invited you to join us in the celebration of the first centennial of an epoch in our history that is not paralleled or eclipsed by any in the world's history, for it led to the establishment of the predominence of the Anglo-Saxon over the Latin race on the American Continent. I mean no disrespect in the statement of this fact. I simply state it in modest pride, with due respect for all nationalities and especially for the Latin race. When it is remembered what has been accomplished since the purchase of this territory, what remarkable development and building up of cities and wealth between here and the Pacific Coast, you will pardon me for indulging in this self-congratulation as to the power of our country and of our people. And permit me to tell you that we feel all the more proud because it was accomplished by peaceful diplomacy and but partly by destructive wars. The patriotic emotions which the memory of this Louisiana Purchase epoch arouses in our hearts, causes me to entreat you to join with us in our song and jubilee, yea, shout with us, for the glory of a Republic which has become an Empire, without the loss of virtue, liberty or independence. The purchase of this territory was the beginning of a policy which has made us a great republic, and was principally the work of one of our great men, who was a seer in statecraft and a colossus in executive official courage. In the deliberations and discussions of this Congress we are realizing events anticipated by his prophetic vision. If, by the assistance of the deliberations of this Congress we shall open the second century after this important epoch in our history, it will have proved to have been a most auspicious event. It has brought together men of thought and action, pioneers, conservators and promoters of the highest civilization the world has known. The fact that from all points of the compass men are here to participate in the work of this Congress shows that it is an undertaking that has at its bottom as a foundation the welfare of all the people of the world. For we have engineers from all countries, men who have contributed their knowledge, experience and their speculations, brought here to be in a measure adjudged before a tribunal that has not its superior though it is but a voluntary assemblage of but a few years' effort of creation. Therefore, I heartily congratulate you each and all upon this auspicious event, and I express the hope that we will mutually understand each other, and, when we have concluded our labors, we shall have promoted the cause of engineering, that we shall have strengthened every brother of the profession present, not only in the requirements, but in the firm belief that each one has a most exalted mission to fulfill in the world's history. We, as members of the profession, however earnest, thoughtful and industrious, too often lack enthusiasm, trusting to desirable results following from reasonable effort. That is sound philosophy, but success does not always crown reasonable effort. Enthusiasm and extraordinary effort is too often the price of success and victory. We confidently expect when the work of this Congress shall have been completed our fondest hopes will have been realized, and that, when it shall have become a part of the engineering literature, it will prove a depository of ten years of progress in our profession, in which there has been accomplished as much, if not more, than in any preceding decade. Therefore, I congratulate you upon this auspicious event.

Mr. D. H. Francis, the President of the Louisiana Purchase Exposition was to have addressed you next in order. A death in his family prevents his attendance. Mr. F. J. V. Skiff, Director of Exhibits, will respond in place of Mr. Francis.

Mr. Frederick James V. Skiff.—Mr. President, Ladies, Members of the Congress and Visitors; it is a difficult task under any circumstances to appear to the satisfaction of others in the place of President Francis. I, usually, in fact, always, regret if circumstances compel me to do so. Therefore, I regret it this morning, because as your President has said, Mr. Francis is absent on account of the death of his uncle, who stood much closer to him than that relationship usually does.

However, the exposition management could not permit an assemblage of this distinguished character to gather together without some expression of its unusual gratification. The Congresses and different International and National gatherings that from time to time are meeting at this Exposition number in the total about four hundred, and of how much greater value this register of universal opinion and judgment and thought will be than the mere exposition of material things, you, gentlemen, are as capable of judging as we are, we think, of estimating. This Exposition, I think, will appeal to all students, scholars, professional and technical men, as having a higher nature or educational mission than any similar undertaking. In fact, it is the basis of our labors and the hope of our toil. The Exposition in a material sense will pass away to-morrow, in a day, and the memory of contemporaneous people will sustain it as a record for only a short time. But the record of the thought that created the Exposition, not referring to the management, but the record of the thought through which these material evidences of man's condition and society's condition at this time—that record becomes an asset that will build and sustain the ages.

Unprepared, as a man should be to address a meeting of this nature, as I am, I shall not presume, ladies and gentlemen, to do more than extend you the cordial welcome of the management. An opportunity of this character, to say something to receptive minds, to get men of thought and push and progress in sympathy with what you may have entertained with reference to the affairs of society at this time, and of the importance of professions in its development, is an occasion that I would have welcomed very warmly. But, as I say, in justice to you and in fairness to myself, I must not follow in a rambling way the fugitive thoughts that occur to me. I have just this moment received the suggestion that I appear in behalf of the President, and I left an important committee for that purpose. However, no matter how accidental, and, under the circumstances, how painful my appearance here this morning may be, I desire to confess to you the honor I experience in addressing such a gathering. In behalf of the management, let me assure you that the Exposition appreciates a meeting within its domains of such a magnificent company of people, both in its personnel and in its relationship to life. We see the evidences of your accomplishments everywhere about these grounds. It is almost trite to say that there would have been no progress without the engineers. We thank you for being present. We highly appreciate the representative character of this meeting. We trust that your deliberations will be attended, not only with the intellectual and scientific results that you aspire to, but to your own personal comfort and satisfaction, and anything that this Exposition can do to contribute to that end, let me assure you on behalf of the President, the Exposition is entirely at your command.

The Chairman.—The next thing in the way of an address comes from a gentleman who has been amongst us but a few short weeks, less than a month, but, in that brief interval of time, his association with us has endeared him greatly to us as a people and particularly as engineers. The birthplace of such a man cannot be wrong, no matter upon what spot of the globe it may be found. England is proud of him, and, had his place of nativity been in the United States, though he could not have contributed perhaps as much as he has to England, it would nevertheless have made us prouder and endeared him to us all the more. It is not necessary for me to introduce Sir William White to this audience. He is too well known. Sir William, will you kindly take the stand.

Sir William H. White.—Mr. President, Ladies and Gentlemen: It is impossible for me to express my sense of obligation to you, sir, for the very kind terms in which you have spoken of me and of my work, and I rise, not to refer to any personal work or merit or demerit of my own, but at your request to speak on behalf of all foreign engineers taking part in this Congress. To be asked to undertake that task is a very high distinction. This Congress has brought together, as you have said, sir, engineers of eminence from all parts of the civilized world, and its proceedings will contain contributions from men who have distinguished themselves in the several branches of their profession and who here put before this Congress their matured thoughts and the record of their experience. And to speak on behalf of such men before such an audience is an honor that I most highly appreciate. I suppose the fact that I have been asked to do so is chiefly due to the circumstance that this year I am the President of the Institution of Civil Engineers, and it is to me a great happiness that in this year of office it has been marked by many new departures and has led me to come to the United States for the first time in my life and here as the representative of what is the parent Society of Engineering in all English-speaking countries of the world to join hands with our American brothers. And I am sure I may venture to go one step further and to say that all the foreign delegates present here from all other countries heartily share in the feeling of regard and respect which I have endeavored to express on behalf of the English Engineers toward the American Society of Civil Engineers, whose guests we are on this occasion. The Institution of Civil Engineers is trying to the utmost of its power to show the respect which it has for the American Society of Civil Engineers, and, as I ventured to say at our first meeting in New York when you were so good as to receive us, we have done in your honor that which has never been done before in the eighty-six years of its existence, and we have done more for this Congress than we have ever done for any Congress, even including that great Congress held in Glasgow, which was also International. There the Association in its corporate capacity had no appearance. The President of the Institution was President of the Congress and nearly all the Presidents of the several Sections were members of the Institution; but the Institution in its corporate capacity did not appear then. But now, to-day, by the decision of the Council and with the approval of the members, I stand here, sir, to convey to you the good wishes of my friends and fellow members who are present here in numbers to-day, and behind them the good wishes—the best wishes for your success herein of all your engineering work—of the eight thousand members of our Institution, and amongst those eight thousand members there are many of the most eminent men who are citizens of the United States and prominent in your Society. It is a great joy to us to think that wherever we have gone on this continent we have been met by members of our Society, and so we have felt even more at home, because when a man comes to you and you clasp hands with him, although you have never seen him before, there is a common bond, a common interest and sympathy that is difficult to express, but which is none the less real and true. We are old, but we are by no means effete. We are vigorous, which is shown by this long journey we are taking in our old age; and in many directions we are showing signs of life. During the last two or three years steps have been taken by our Institution which mark new departures, new departures which I venture to say, and I say it with all respect, will be followed by other engineering Societies.

We are proud to speak of engineering as a profession. So it is. It is a learned profession, or should be, and the Institution of Civil Engineers has marked its appreciation of that fact by insisting upon a strict educational test embracing engineering education and scientific training in all departments. In the matter of standardization, a thing which affects manufacture and production in a most remarkable way, the Institution has taken steps to appoint a most powerful Committee which has been at work now some considerable time, and many of the reports of which have been placed at the disposal of your Society and will be found here. We may not be able to standardize throughout the world, that is not to be expected, but we are glad to place at your disposal such information and facts and conclusions as we profess and we ask you for an interchange, feeling sure that the experience of this great Continent in its various branches of engineering will be of the greatest value to us.

And then this year, we have taken another step which naturally grows out of that of which I spoke just now, the insistence upon an educational test, and we have again appointed a Committee representative of the Great Engineering Society of Great Britain to deal with the question of engineering education, a matter which has been most thoroughly considered here and where we have been able to learn much and which we all agree must largely affect the future of engineering.

These are only samples of what I venture to say are evidence of vigorous life in an Institution which will soon be ninety years old, and giving me, as President of that Institution to-day, the prominence you have in asking me to respond for your foreign visitors, may I again say that I take it not merely or chiefly as a personal matter, but as a tribute to the dignity of the office which I at present hold.

This gathering, as we all know, is one of a great series. I suppose we may say that the first great occasion of this kind was at Chicago eleven years ago, and since that time there have been similar gatherings in Paris, Glasgow and Düsseldorf, and naturally we come back to this country and the cycle begins again, and it could not begin under happier circumstance or in a more suitable locality.

You have spoken of historical events connected with St. Louis.

To us these are matters of great interest because they had to do with the growth of this great kindred people, but, if I may say so, I think I voice the feeling of English Engineers when I say we cannot come to St. Louis without thinking of Capt. Eads. To most of us he is now only a name, but I think all of us will make a visit to the levee and look upon his great bridge standing there. And we all know of those gigantic works he carried out at the mouth of the river and I remember when only a young naval architect of reading with immense interest the account of what Capt. Eads did in the way of the construction of a flotilla in these waters during the Civil War, a work accomplished under enormous difficulties, under great pressure of time, but done in a manner that fully met the requirements of the situation, and which no doubt had great effect on the results of operations in this region.

These are only a few instances of what we remember of Capt. Eads, whose name I am sure is cherished as that of a great engineer of whom we are all proud and whom we all respect.

And then as to the organization of this Congress. You, sir, on this side having faculties for organization and you had the experience of the Chicago meeting to go upon besides the knowledge that had been acquired from these other gatherings in other countries, but if I may say so, anyone who looks through your *Proceedings* and sees the list of papers prepared by the many authorities can want no further evidence of the capacity, the grasp and comprehensiveness of the scheme which has been laid down for our meeting here. But there is one little touch which I shall venture to mention. I came into the room this morning and saw this diagram on the board giving the particulars of the places of meeting of the various sections. I had, before I saw that scheme of the places of meeting, the greatest admiration for the skill that had been displayed in the organization, but the final touch was reached when I saw that Section G, the Naval and Military, worthily represented by Major Craighill who sits here, was to meet in the room of the Lady Board of Managers. I do not remember any gathering with which I have had to do heretofore where I have seen the proprieties so beautifully dealt with as in Section G, and I am sure with Gen. Craighill at our head, those of us who belong to G will have a good time. Section G is reserved for us. It is only those that belong to the naval and the military that are to join the Lady Board of Managers.

Now, resuming what I have to say about this gathering it is most fitting that these Congresses should be held in connection with such expositions as we have here in St. Louis, and I entirely echo the sentiment expressed by the gentleman who preceded me when I say that on the educational side this Exposition certainly bears a part. It is a proud thing for an engineer to walk about these magnificent buildings, to see the wonderful collections there and to think that he belongs to a profession which has had so much to do with the creation of all these marvels and with the great advances that have been made in every direction for the benefit of mankind. One has to try to cultivate a spirit of modesty because there is such a danger as one sees what has been done of pluming oneself too much and so perhaps becoming ineffective for the future, for a man, in my experience of men, who thinks he has finished learning had better retire from the profession and take something less difficult than engineering. Here we have the latest applications of science, engineers of all countries contributing not merely to show what is the present position of affairs, but to give the means of indicating what are future tendencies in the several branches, and then what is in evidence so much and what rejoices us that we cannot walk through the Exposition without thinking that the old ideas of secrecy and the monopoly in invention is gone and instead we have the modern spirit in which invention legally protected is published to the world. It stands upon its merits and its results decide whether or not it shall be largely utilized. And to meet here with such a gathering as this, a gathering of engineers of eminence from all parts of the world, it is a lesson and not only a lesson, but a great opportunity to all of us. Science and engineering are necessarily and inevitably connected. Their union is indissoluble. I for one refuse to admit that applied science is in any way inferior to what is called pure science. I have the highest respect for a man who devotes his life to a service without thought of commercial result or pecuniary reward; but I say, as I have often said before, that to my thinking a man who takes upon himself the task of doing things—enormous responsibility, not merely for material benefit, but for human life—that man is equally worthy of respect and admiration, and when I hear, as I sometimes do hear, men hold up the example of Faraday and say that is the noblest character who makes discoveries and puts aside the question of personal advantage, I say Faraday is a noble type, but a man who, like George Stephenson, faces the unknown and carries through enormous undertakings for the benefit of mankind is also entitled to equal respect from his fellows.

The engineer must be a man of science. A purely scientific man, devoting himself as he does to tasks commanding the highest grade of the intellect, is free from many cares that press upon the engineer. Let us then go our several ways respecting one another, helping one another, and we may be sure that for that feeling of happiness the world may be better.

And then, as one goes about the Exposition, there is another thing to be seen and that is the marvelous diversity under modern conditions of the tasks of the engineer. There was a time when the Civil Engineer, in his own person, could embrace all kinds of engineering, and did so. He practiced nearly every kind and every branch of engineering. All branches, whether canals of the kind that belong to the end of the Eighteenth Century, or railroads, or bridges, all lay within the grasp and practice of one individual. But how different is it now! We may have some knowledge of many branches, but we all find as the years go on that we become specialized whether we will or not. It is inevitable that it should be so. As in science, so it will be in applied science or engineering. So let us remember always that all branches of engineering are inter-dependent, they are all related. The lines of demarkation cannot be defined. A man may intend to be one thing and come to be quite another. I never intended to be an engineer at all, but somehow I came to be one. I had no one belonging to me that had the least connection with shipbuilding, but I have been a shipbuilder. I suppose it would be called the force of circumstances, but so it happened, and in engineering as a whole men find their place. I do not mean to say they continue all their lives to practice one branch of engineering, but the unavoidable necessities cause men however able to be attached to some special branch of work as the years go on and to distinguish themselves there. And so in this Congress, with its several sections, let us feel that although that is true, and specialization is unavoidable, let us remember that we have the common tie of all being engineers, and that we help one another, that no one stands alone but even the specialist is dependent upon his brethren.

Now, if I may refer to that which I know most about, shipbuilding, just think what the modern ship involves. The modern ship could never be but for the joint action of many branches of engineering. You know you have the Mining Engineer, the Metallurgical Engineer, the Mechanical Engineer, the Marine Engineer, and the Electrical Engineer, all contributing to help the Naval Architect to produce the ship that plows the sea, and defends the honor of the flag. The Naval Architect takes the concrete responsibility, but he avails himself of the skill of his colleagues. And so it is in other ways, and one might multiply illustrations. The truth is we are not divided, although we must be specialized, and that, I think, is the great feature of our Institution and of your American Society that you do not exclude any branch of engineering from membership, that a gentleman who is accomplished and competent in any branch of engineering, is with us and I believe also with you, entitled to the privileges of full membership. I have sat in the chair at the Institution of Civil Engineers during the past year and listened to discussions of the most diverse character, and I have never on a single occasion failed to get some good, some idea, some suggestion, some additional information that would be helpful to me in my own work. And then, if I may go one step further, I like to think on this occasion how one nation helps another, not merely how one department of engineering helps another, but how one nation helps another. We, in England, have grown to be the great shipbuilders of the world. We are proud of it. We claim it, we know we hold it, and we intend never to sacrifice it as long as we can hold it, but we do not forget that we owe our knowledge of the theory of naval architecture to our French brothers. The French, in naval investigation, were leaders, and they to-day remain in the forefront of investigation, and we, who learn so much from them and who now endeavor to equal them on the theoretical side, I think, in our turn, have been able to give some service in the department of the practical art of shipbuilding. And so in America. I like to think of what I claim we have in England, in "the old country," of some of the things which are reckoned as belonging so peculiarly to the United States. Take, for instance, the America's Cup. Everybody knows that the America, which took the cup, was designed by a gentleman named Steers, who was born in the United States. That you all know. I am bound to say you can all tell me that, but I wonder if you all know, as I do, that the Mr. Steers who designed the America was born a week or two after his mother landed here, and that his father was born in our own little town. I will leave it to you to say how much claim we have on Mr. Steers, and I shall not enter further into that discussion.

Now, of course, in relation to Engineering in its great development we can fairly claim that we have done most, as was said by the gentleman representing the Exposition, we have done most—I think it might be more modest to say much, toward all the modern changes that have been beneficial to humanity. It is extraordinary when one thinks of what work the engineer has done in the promotion of the knowledge of one nation by another in the maintenance of peace, in the development of the forces and resources of Nature, in all that makes for the good of Mankind. And that is the side, I think, we should be most inclined to dwell upon, the benefits that engineering confers upon our fellow-creatures, the waste places of earth made to blossom, distance annihilated. My life has been largely connected with the construction of warships, and perhaps it may seem a funny thing for me to say that that is the side of engineering of which we should be most proud and I am going to make a claim that will astonish you. I am sorry to deal with a personal matter, but I cannot make my point without doing so. It has been my fortune to spend more of the public money of England upon naval armament than any man who ever lived, many times over. After spending $500 000 000, which was about the expenditure that I had to be responsible for during my tenure of office, upon warships and armament, it may seem strange that I should lay claim to having done much for the maintenance of peace, but I do make that claim, and I do it for this reason: There is an old Latin proverb which we always had before us at the country school at Portsmouth, "*Si vis pacem para bellum,*" "If you want peace be ready for war," and I can say, personally, that the possession by England of a great navy has been the cause of the maintenance of peace on many occasions, when otherwise peace would not have been kept. The policy of England and of the United States is that of peace, but no one can hope for continuous peace, as the world is, who lies open to attack which cannot be met, and so I venture the claim that even on the war side the same might be said of military matters. There, the engineer is just as essential as in the building of warships, whether it is the production of guns or ammunition or explosives, whether it is the production of the means of transport, including the latest development, the traction engines, which we used so much in South Africa, or the electrical equipment, a modern army is impotent without the work of the engineer, and there, again, while I deplore that it should be necessary that these large sums, which could be so much more usefully employed in developing peaceful industries, should have to be expended in warlike preparations, yet as the world is, taking human nature as it is, the engineer who devotes himself to warlike work, in my judgment, can fairly claim to stand among the peace-makers.

Ladies and Gentlemen, I fear I have been too long, but I do wish before I sit down to go back to that idea, that we engineers throughout the world are brothers in arms fighting against the adverse forces of Nature, turning them into useful forms, taking the great waterfalls of this country and harnessing them into the service of Man. We have just been to Niagara, and there on both sides we saw those wonderful works proceeding which will turn those magnificent falls into the service of Mankind on an even more extensive scale. In Toronto I was speaking to that great English professor, Goldwin Smith, and he said to me "If you are going to Niagara, I hope you will say to them 'don't let that beautiful wild spot be spoiled by great engineering operations.'" When I got to Niagara I found that the Superintendent of the Canadian part was a Civil Engineer, a member of our Society, and there was the engineer, and one of his great preoccupations was that when those works are completed Niagara shall be as beautiful as it is possible for human skill to make it. That is a part of the scheme. It is not necessary to make things ugly to make them useful, although some people act at times as if they thought it was. That is the idea, brotherhood of engineers, brothers in arms. I like to think of Engineering as a great and splendid tree. Its root strikes deep into the thoughtful soil of Science. Its main trunk lies in the idea of subordinating and utilizing the forces and products of Nature in the service of Mankind. Its branches ramify in all directions, and are continually throwing out fresh shoots as new demands arise and new things have to be met, and behind it all lies the fundamental idea that all Engineers have one common object, the service of Humanity.

Chairman Hermany.—I will now call upon our foreign representatives to say a few words upon this occasion. We sincerely hope that they will respond in the order called upon.

The first gentleman who will address you is M. Loicq de Lobel, Delegate of the Société de Geographie, of Paris.

M. Loicq de Lobel.—Mr. President and Gentlemen of the International Engineering Congress: You know with what difficulty a Frenchman attempts to speak the English tongue correctly; and, therefore, you will permit me to use my mother tongue to express to you the great pleasure and satisfaction that we have to be with you because you are noble representatives of the "Genie Civils" of all nations, and I can say without fear of contradiction that the greatest works of the past century are represented in this body by those who were their inspirers and executors.

I have visited some of your Universities, and have seen what you are there accomplishing through improved methods in instructing engineering students to combine theory with practice.

I am sure that this Congress will produce good results in the development of the "Genie Civils" and in cementing a more intimate union between all engineers of the world.

I hope that you will find early another occasion to hold such a Congress, next time in France, when we shall make efforts to give to you the same cordial reception that you have accorded to us here to-day.

Chairman Hermany.—The next gentleman who will address you is Herr Oswald Ehrlinghagen, of Germany.

Herr Ehrlinghagen spoke in his own tongue, and his remarks were not reported.

Chairman Hermany.—You will next hear an interesting address from a distinguished citizen of our sister Republic, the Argentine Republic; a gentleman who has grown honorable and eminent in the pursuit of the great Profession of Engineering; one who will impress you as one of the veterans in this grand profession. I have pleasure in introducing Hon. L. A. Huergo, of Buenos Aires.

Mr. L. A. Huergo.—Honorable President, Ladies and Gentlemen: To voice the importance of this Engineering Congress of the World's Fair held at St. Louis is useless, because your own presence here testifies sufficiently to it, you who have come from every part of the world to take part in and to evince your interest in this International Congress of Engineers, and to witness at the same time the results of the accumulated and combined effort of Humanity as displayed in this Exposition.

I have come from the Argentine Republic, and have been traveling I may say two months, for I stopped in London *en route*, and from there had to come through Canada to reach New York, and finally to the St. Louis Exposition.

I cannot prophesy what the results of this Congress will be, because my time here so far has been given to learning what I could, and I have had no time to form conclusions.

I have been given, through the courtesy of the American Society of Civil Engineers, at New York, the privilege of inspecting some of the principal engineering undertakings there. I have visited the Subway, the Navy Yard, the West Point Academy, and some of the engineering shops, electrical and otherwise; and I have admired the great kindness and complaisance with which I have been everywhere received.

have collected a great deal of information about the transportation questions which are so all-important throughout the whole world to-day, and upon the practical solution of which depends every industry.

I have had great pleasure in visiting this gathering, composed as it is of so many English, German and French engineers, as well as those of other nationalities; and I believe that the Engineering Profession combined hold within their hands greater powers and possibilities than did the Olympian Gods of old.

I have had the great pleasure—which alone would have compensated for all the labors of my travels—of shaking hands and exchanging a few words with Thomas A. Edison, and I consider that such a man, who has helped us to achieve that which a few years ago would have been accounted impossible, is worthy of a place higher than that accorded to those pagan gods.

I have been to Washington; and am under great obligations to the Department of War, to the Department of the Navy, and to the Agricultural Department, for their liberal information as to engineering as applied to those various departments,

You will consider it rather strange that I should address you in the English tongue. I was in fact educated in the United States, but so long ago—forty-seven years, almost half a century—that I am quite at a disadvantage in so addressing you. You may wonder then, why I choose to use English rather than Spanish, in which latter tongue I would not be so hampered to find a proper vocabulary; it is simply because were I to use Spanish I would be understood by none, and therefore I have used English. One of my reasons for so doing is that for fifty years I have followed the great maxim of George Washington: "Do what is right, and fear no man." (Applause.)

Chairman Hermany.—I call next upon Mr. B. Bachmetew, of the Polytechnic Institute of St. Petersburg, Russia, whom I take pleasure in now introducing.

Mr. B. Bachmetew.—Mr. Chairman, Ladies and Gentlemen: I feel greatly honored in appearing before you as the representative of Russia, and as my command of English is very limited, my remarks will be brief.

One fact, especially, I wish to impress upon you, and that is that it is subject for congratulation universally that the central idea which has inspired this Congress is so fully exemplified, and that we here see so fully illustrated the fact that the science of Engineering is growing more and more international in scope, and that engineering progress is developing more and more upon these international lines.

I further wish to convey to you from the Engineers of Russia a hearty greeting and best wishes. (Applause.)

CHAIRMAN HERMANY.—The next speaker who will address you is Mr. Otto F. Schoszberger, C. E., of the Imperial Technical High School of Vienna, Austria, whom I have the honor to now introduce.

MR. OTTO F. SCHOSZBERGER.—Mr. President, and Members of the International Engineering Congress: As a representative of the Engineers and Architects' Association of the Technical High School of Vienna, I bring to you hearty greetings from your foreign co-workers. I am rejoiced to participate in the deliberations of the International Congress of Engineering in St. Louis, under the auspices of the American Society of Civil Engineers, so grandly planned, organized and carried out in all its details. I find it to far exceed my anticipations.

CHAIRMAN HERMANY.—The Secretary will now make some important announcements.

SECRETARY HUNT.—I have just received the following cablegram:

"LONDON, October 3d, 1904.

"*Secretary Engineering Congress,*
WORLD'S EXPOSITION, ST. LOUIS, MO.

Greetings Junior Institution Engineers. Best wishes. Success.
"MOULTON, *President.*"

I wish, further, to announce that Colonel C. M. Watson, Commissioner-General for Great Britain, has issued the following invitation:

"The Commissioner-General for Great Britain and Mrs. Watson request the honor of the company of * * * at the British Royal Pavilion on Wednesday afternoon, October the fifth, 1904, from three to five o'clock, to meet the President and Members of the International Congress on Engineering."

I am requested by Colonel Watson to say that owing to the location of the building in the center of this Fair, it will be necessary for each member of the Congress, and each of the ladies who are to attend, to bring with them a card of introduction which will be issued. Those who have registered will a few hours later be able to receive those cards at the Secretary's office in this building. (Applause.)

I have the following letter:

"Mr. CHAS. W. HUNT,

"*Secretary International Engineering Congress,*

"HALL OF CONGRESSES, WORLD'S FAIR.

"DEAR SIR: The Engineers' Club of St. Louis extends a very cordial invitation to the members of the Engineering Congress to attend a smoker, Friday evening, October 7th, at 8.30 o'clock, in the Missouri Building, World's Fair grounds.

"It is the hope of the Club that all the members of the Congress will find it convenient to be present that evening.

"Very truly yours,

"R. H. FERNALD, *Secretary.*"

On this occasion it will not be necessary to have tickets; the badges of the members will admit them. Nothing is said about the ladies, so I presume that they are not expected.

I also have the following:

"ST. LOUIS, MO., Sept. 30, 1904.

"CHAS. W. HUNT,

"*Secretary of the American Society of Civil Engineers,*

HALL OF CONGRESSES, WORLD'S FAIR.

"DEAR SIR: In behalf of the Local Committee of the American Institute of Mining Engineers, we gladly extend to your Society and other visiting engineers, the use and courtesies of our headquarters at Block 74, Mines and Metallurgy Building. All Engineers are welcome, and we will be glad to extend any courtesies within our power.

"Yours very truly,

"H. A. WHEELER.

"Local Sec'y."

Also the following:

"ST. LOUIS, MO., Oct. 1, 1904.

"Mr. CHAS. WARREN HUNT,

"*Sec'y American Society Civil Engineers,*

ADMINISTRATION BUILDING, WORLD'S FAIR.

"DEAR SIR: The Goldschmidt Thermit Co. has offered to make special demonstrations for the members of the International Engineering Congress on such days and at such hour as may be most convenient. The Company demonstrates the application of melting and smelting steel by chemical reaction, and I believe it would interest a good many engineers to see this new process. Perhaps it would be well to have this demonstration given on two different days. They are executed in front of the metal pavilion near the Mines Building, and take fifteen to twenty minutes. I would suggest Tuesday and Friday afternoons, at 4.00 P. M., and would be pleased to hear from you, etc.

"Very truly yours,

"O. E. Mogensen."

Mr. Mogensen suggests Tuesday and Friday afternoon, at 4.00 P.M. I think we had better adopt these times, and if those members who are interested in the matter will be at the Metal Pavilion, near the Mines and Metallurgy Building, at 4 o'clock either to-morrow or Friday afternoon, they will be given this opportunity to witness the experiments named.

I am requested by the Canadian and Niagara Power Company to say that there are thirty copies of a little pamphlet which was given to the British Engineers on their way through Niagara, and which copies have been forwarded here for delivery by me to those members of that body who did not go on the train. They can be had by application at the office of the Secretary.

You all have copies of the programme, of course, so I do not think there is very much to call attention to there except, perhaps, to explain that it is the intention to carry out the programme as printed. If the discussion of any one of the subjects assigned for Tuesday is not finished it will not run over until the next day, but will be called up at the last meeting, which has been left open for the purpose.

The meetings of the Sections are shown on the bulletin-board, which gives the location of the various meeting-rooms, and it will not be at all difficult for you to find them. However, inquiry can be had at any time at the Secretary's office, which will be open at all times, and you will be directed properly.

Mr. President, Mr. Brooks, of Boston, wishes to say a few words.

Fred. Brooks, M. Am. Soc. C. E.—Mr. President and Gentlemen: I wish to get a little information apropos of the fact that many of the members of this Congress are going home eastward, and to those who can make it convenient to stop in Boston, I would say that in Boston there is a large local Society, the Boston Society of Civil Engineers, in behalf of whose members I stand here to say that they wish to afford facilities for the members of this Congress to see what there is of interest in Boston. There is a great deal of interest in Boston. We assume that the world knows that to be a fact, but the information I desire in order to afford proper opportunities for showing any of them, is as to how many are to go, and when they will be there; and, as far as may be, what they particularly desire to see. A list will be opened in the Secretary's office after this adjournment, and those who have time to go to Boston will please make this known to the Secretary, and through him to me.

SECRETARY HUNT.—Before adjournment of this meeting of the Congress, I would like to make an announcement in connection with the Annual Convention of the American Society of Engineers.

The first meeting of that Convention will be held in this hall at 2.30 P. M., to-day, at which meeting the President, Mr. Hermany, will deliver the Annual Address.

At the close of the address of the President there will be a Business Meeting of the Society. I should like to say that all members of the Congress will be very cordially welcome at the meeting at which Mr. Hermany delivers his address.

CHAIRMAN HERMANY.—Gentlemen, this first session of the International Congress of Engineers now stands adjourned until the final session to be held on Saturday at 10 A. M.

CLOSING SESSION.

October 8th, 1904, 10 A. M.

October 8th, 1904, 10 A. M.—Mr. Robert Moore, Past-President, Am. Soc. C. E., in the chair, Chas. Warren Hunt, Secretary.

THE CHAIRMAN.—The meeting will please come to order. I am very sorry, as I am sure you all are, at the announcement that Mr. Hermany is indisposed and may not be able to be here this morning, although we are not wholly without hope. It has been thought best that the meeting should be called to order in his absence. I will, therefore, ask the Secretary for the next business.

THE SECRETARY.—The Convention was to have a report from each of the sections of the Congress and I have one from Mr. Alfred Noble, Chairman of Section A, who is not able to be here this morning.

The report was read by the Secretary, as follows:

REPORT OF THE CHAIRMAN OF SECTION A.

"I beg to submit the following concerning the work in Section A:

"The several subjects referred to this Section were treated in an able manner by the engineers assigned to prepare the papers. The present methods adopted in Harbor Development on Sea Coasts in the British Empire, in Holland and France were fully described, as well as the methods in use on the Sea Coast and on the Great Lakes of the United States. Much interesting discussion was elicited, both written and verbal. Two papers, treating specifically of the use of concrete blocks in Harbor Works, one general in character, the other referring to a specific case in Japan, were received too late to be printed for the use of the Congress.

"On the subject of Natural Waterways an excellent description was given of the methods successfully adopted for the rivers of Holland, and in the discussion a *résumé* was given of the experience gained in the improvement of the Channel of the Mississippi. A description of the so-called Rolling Dams was given and discussed at length, the discussion eliciting an interesting comparison of rolling and bear-trap dams.

"The subject of Artificial Waterways was treated with reference to three important Ship Canals of Holland, and the Inland Navigation System of France. A comprehensive paper on the general subject of Artificial Waterways was presented and special reference made to the Manchester Canal and suggested waterways across the Island of Great Britain. Typical canals and canalized rivers were also fully described in a paper relating specifically to those topics.

"The subject of the concurrent development of traffic on waterways and railroads was taken and examples cited to show that they were concurrent. This view was also supported in the discussions.

"The several papers on Dredges described comprehensively the various kinds found best adapted for different conditions on this continent, as well as in other countries. Tests of the capacity and efficiency of dredges, carried out in much detail, on the Mississippi elicited very full discussion.

"Under the head of Wharves and Piers the methods adopted in New York Harbor received adequate treatment in a descriptive paper and in the discussion which followed.

"The meetings of the Section were well attended. The papers and the discussions were on broad lines, and it is believed that the proceedings of the Section will constitute a valuable portion of the records of the Congress.

"Very respectfully,

"A. NOBLE."

THE CHAIRMAN.—Have you record of Section B? Is Mr. Croes, the Chairman of that section, present?

The Secretary reported, in the absence of Mr. Croes, Chairman of Section B, that four meetings were held of Section B, at which 8 papers were presented. There were also 11 discussions in writing and 38 Members participated in the oral discussion. The average attendance was 31.

Mr. Hermany, the Chairman of the Congress, arrived and took the chair.

THE CHAIRMAN.—We will now call for the report from the Committees of Section C.

MR. MOORE.—Mr. President, I have no written report on Section C, but I will say that we had three meetings, first on Railway Terminals, in which the very important question, and one which is pressing hard on all the trunk lines in all the great cities of railway and passenger terminals, was quite fully discussed. We had the general statistics presented by Mr. Corthell, and a paper by Mr. Foxlee, of London, describing the present practice in London and the proposed modifications at one of their great stations. Mr. Pontzen also described some of the most recent devices for handling passengers and baggage in the City of Paris.

The second discussion was on Underground Railways, a paper by Mr. Wm. Barclay Parsons, descriptive of the practice in New York and of the present elegant subway which is ready for use, another by Messrs. Basil Mott and David Hay, giving their views on the subject.

In the absence of Mr. Parsons we were obliged to discuss his paper without him.

The third topic for discussion was Live Loads for Railway Bridges, and that was fully discussed and some very important suggestions made. Then the Ventilation of Tunnels, which was presented in a paper by Mr. Charles S. Churchill, of the Norfolk and Western Railroad, and Mr. Francis Fox, of London, both of whom showed by actual example the entire feasibility of rendering even the longest tunnel perfectly safe and wholesome, and showed, also, by figures, the the great economy of operation accomplished by so doing. We were also fortunate on that day in having present Mr. Parsons, who gave an important and interesting account of some matters concerning the subway and also the ventilation of tunnels which was the subject proper for that time.

The meeting then adjourned *sine die*. This was earlier than some of the other Sections adjourned, but I think it might be expected that those who go by rail should arrive earlier than those who go by steamboat, canal or other antique method. I will say, however, that these meetings were well attended. The discussion was interesting and valuable, and I think the members of Section C will go back with the strong impression that the meeting was very well worth their coming here to attend and that it will very evidently impress remembrances.

THE PRESIDENT.—The next report is from the Chairman of Section D, Mr. Stearns.

MR. STEARNS.—Ladies and gentlemen, I have not prepared any detailed report of the operations of this Section; in fact, it would be very difficult. It was so largely attended and there were so many discussions that it would be rather tiresome to enumerate them. The subject was Materials of Construction. On the first day, the papers on the manufacture of Steel and the Manufacture of Cement, were submitted and very thoroughly discussed. At the next meeting the discussion was on Concrete and Concrete-Steel, and it brought out a very thorough and complete discussion which could not be finished on that day. The third day we took all the Materials of Construction, including Steel, Cement and Wood. These also were fully discussed. Then on the fourth day we continued the discussion of Concrete and Concrete-Steel and that will develop a great deal of interest. These results in this section of the Congress will make valuable literature. When I first came to St. Louis I looked in the paper and in the funny column there was the story of a gentlemen who had seen on the programme that there were congresses at which there were men and women, that he had attended the congresses, and that he had found that the men with the brains were on "The Pike." That is not true of the engineers—

I don't mean that they are not men of brains—but it is not true that they were on "The Pike." I was surprised that with all the attractions there were at the Fair that there should be such an attendance at the Congress, and as there was at the section over which I presided, and, as I hear, by report, were at the other sections. It shows the interest of the engineers in their profession. Discussions were thoroughly valuable. They came from gentlemen who were prominent in the various lines, and, further than that, they were truly international. On every subject that was taken up we had very interesting discussions from men from the various nations who were well qualified to discuss the subject. It seems to me that those who organized this Congress and those who planned it so as to bring forth all these papers and discussions should be congratulated and worthy of great praise for the work they have done.

THE PRESIDENT.—The next report is from Section E, Mr. H. S. Haines, Chairman, and the Secretary will read his report.

REPORT OF THE CHAIRMAN OF SECTION E.

"The papers presented in this Section were upon topics which attracted a considerable audience at each session and elicited interesting discussions, to which valuable contributions were made by engineers from England, Germany and Sweden.

"The topic of Pumping Machinery in connection with municipal water supply was discussed throughout one session and carried over to another. The papers upon recent locomotive practice were replete with information and contained some valuable suggestions as to the field for improvement.

"The papers upon Passenger Elevators and upon recent tests of Steam Turbines, in connection with the discussions upon them, represent the views of experts with reference to the prospective field for such devices that are based upon the most recent experience.

"The total results of the four days' sessions in this Section have been of a character to confirm the opinion that, for an International Congress to be of real utility, its discussions should be confined to a few live topics, presented in papers by leading experts and discussed in appropriate sections upon appointed days.

"H. S. HAINES."

THE PRESIDENT.—The next report is from Section F, Electrical, and the Secretary will read that report.

THE SECRETARY.—The Chairman of this Section, Mr. Geo. H. Pegram, M. Am. Soc. C. E., was forced to leave before this time and did not have an opportunity to write a report. I can say, however, that in this section, while there were only three papers presented, they were on topics of great interest, and while the meetings of the Section were not very largely attended, there was an exceedingly good discussion by experts.

THE PRESIDENT.—The next report is of Section G, Military and Naval, General William P. Craighill, Chairman.

General Craighill.—Mr. President, Ladies and Gentlemen: The subject of Section G was Military and Naval Engineering, and it was one not to lead us to expect a very large attendance, and in that our expectations were realized, but I wish to say that while we were not great in numbers, that the men of brain were there. Very few of the authors of papers were present, and we had many papers in our Section on all its subjects and it was truly an international Section, and we had papers not only from men of our own Society and civil engineers of the United States, but we also had several interesting papers from England, Japan, France and Russia, so that I think we may make claim that our Section was an international one. We regretted the absence of the gentlemen whose names you find upon this programme, especially Lieut.-Col. Holden, of the Royal Artillery of England, and also Admiral Endicott, from whom we expected a paper and whose presence we also expected, but he neither came nor gave us a paper, at which we were greatly disappointed.

The proceedings of the Section were opened by the consideration of the subjects of Fortifications and Ordinance. We had a very excellent paper on fortifications from one of our army engineers, Major Goethals, who could not be present, and there was no discussion. There was an excellent paper also on Ordnance read by Capt. Burr, of the Ordnance Department of the Army, who presented his paper and precipitated a very interesting discussion. The principal paper on Naval Architecture was by Sir William White, and there was also an admirable paper compiled at the University of Tokio, the authors of which were four of the professors of that institution.. None of them was present, however, which we very much regretted.

On the subject of Marine Engineering, we had an admirable paper from Professor Durand, of Cornell University, and an exceedingly interesting one in French from M. Daymard, Engineer-in-Chief, of the Trans-Atlantic Company, of France. His paper was of exceeding interest.

Upon the subject of Lighthouses and Other Aids to Navigation, we had a number of printed papers, one by Lieut.-Col. Lockwood, Engineer Secretary of the Lighthouse Board, in Washington. We had also a paper from an English engineer, Mr. Thomas Matthews, of Trinity House, London, and also an interesting and instructive work from M. C. Ribière, Ingenieur en Chef des Ponts et Chaussées, Ingenieur en Chef du Service Central des Phares et Balises, France.

Upon the subject of Dry Docks, we had three papers, one from Mr. Cuthbert A. Brereton, of London, one from M. Paul Joly, of France, and another from Mr. Timonoff, of Russia. None of the authors of these papers was present. The discussion of all the papers was exceedingly interesting. It shows that the men who wrote them were men who knew what they were writing about. They presented their subjects in an interesting way, but in a way to give information bringing out the salient points for our consideration, and the discussion was most interesting, although indulged in by a few. As I said before, our numbers were not great. On the whole, I am sure the perusal of these papers will be interesting to civil engineers, as well as to naval engineers.

In conclusion, I wish to say that we were especially favored in this Section by having with us at every session, and during almost the whole of each session, a gentleman whose specialties brought him to us, and his presence was a source of great pleasure and profit to us. I refer to our distinguished confrère, that eminent engineer and most charming English gentleman, Sir William H. White. (Applause.)

THE PRESIDENT.—The next Section H, was presided over by Mr. Chanute.

MR. CHANUTE.—Gentlemen, it fell to my lot to preside over a rather miscellaneous section, and that illustrated very well the wide scope occupied by the American Society of Civil Engineers. The attendance was not large, varying from eighteen to twenty-seven members, but they were truly international. We had visitors, who took part in the discussion, from Great Britain, British Columbia, India, Switzerland, Sweden, Japan and one or two other countries which I cannot locate without the Secretary's notes. The papers were first on Irrigation. On that we had five papers and a number of written discussions on those papers, originating in France, Holland and Egypt, and covering an account of the present situation on the art which is becoming of greater and greater importance in the United States, and an account of what has been done in Java, Japan, the Hawaiian Islands and in India. The discussions were instructive, and will add considerably to the fund of knowledge on the subject. This took up the first day. The second day we had papers on Highway Construction, Foundations and Mining Engineering, which likewise created a very interesting discussion. On the third day we had papers on Engineering Education. In these we were favored with a number of accounts of what is being done in this country and abroad, and were favored with a most interesting talk from Sir William White. We also took up on that day Surveying, on which there was some discussion. The exact data will be furnished by the Secretary. In the course of our discussion it became apparent there were two points, not provided for as yet by the rules, which it was desirable to bring to the attention of the Convention and of the American Society of Civil Engineers. The first was in respect to the different methods that are adopted in the various countries for the measurement of irrigating water. In some countries the unit is the miner's inch and in some others it is acre-feet, and the meeting passed a resolution suggesting that the American Society of Civil Engineers should open communication with engineering societies in other parts of the world, in order to secure if possible some uniform system of measurement of irrigating water and the rules which are applicable to it. Again, it would seem that there were a great many things that might subsequently occur to the members in attendance which they might wish to express, and it was the desire of those meetings at which discussion of the various subjects brought up might be admitted before the final publication of the report of the Convention, and that the Secretary should announce some time beyond which no such written reports could be received. That, I believe, is substantially what was done by Section H.

THE CHAIRMAN.—The Secretary of the Congress, Mr. Hunt, will now make a general report.

SECRETARY HUNT.—I think perhaps it is better, Mr. President, to answer Mr. Chanute's question at once, and to give notice that written discussions from engineers in America will be received up to the first of December, and not after that, for incorporation with the papers in the published proceedings of the Congress, and that all discussions received from the other side before the first of January, 1905, will be accepted for publication.

The programme as announced at the first meeting and issued to members of the Congress has been carried out, and after the first, or opening, meeting, the Congress divided into eight Sections.

There have been 28 meetings of these Sections, and the average attendance at each of these meetings was about 50.

In the discussion of the 38 selected subjects, 97 formal papers written by prominent engineers, by special invitation, were presented. Eighty-four of these papers were in type at the time of the opening of the Congress, and had already been distributed for the purpose of eliciting discussion.

In addition to these formal papers, 78 communications were presented to the Sectional meetings written by engineers who were unable to be present, and, in addition to this, there were 272 oral discussions at the meetings. All of the latter have been stenographically reported, and, after they have been submitted to each speaker for correction, will be collated and published, together with the papers, and issued to all members of the Congress.

The volume of the product of this Congress will be quite large. A fairly conservative estimate indicates that the proceedings of the Congress will cover some 3 500 pages of type, or 6 full volumes of the *Transactions* of the American Society of Civil Engineers.

The total membership of the Congress is between 3 400 and 3 500.

As to its International character it may be stated that in the formal papers presented, as well as in the 78 written communications before mentioned, 11 foreign countries are represented, and furnished about 50% of all papers received.

In this connection, an analysis of the total attendance at the Congress may be interesting. The total number of members and ladies of their families registered was 876, distributed as follows:

North America:		
Canada	9	
Cuba	4	
Mexico	4	
United States	724	
		741
South America:		
Argentine	7	
Brazil	2	
Chile	1	
		10
Europe:		
Austria	3	
Denmark	2	
France	4	
Finland	5	
Germany	9	
Holland	1	
England	70	
Ireland	1	
Scotland	2	
Hungary	3	
Europe (*continued*).		
Russia	3	
Sweden	6	
Switzerland	2	
		111
Asia:		
India	5	
Japan	4	
Malay States	1	
		10
Australasia:		
New South Wales	3	
Victoria	1	
		4
		876

SUMMARY.

North America	741
South America	10
Europe	111
Asia	10
Australasia	4
	876

There has been little time to prepare a detailed report of the work accomplished by the Congress, but, from what has been said by the Chairmen of the various Sections, and from the figures just given, it seems safe to presume that the result will be of such value to the Profession, both at home and abroad, that neither the American Society of Civil Engineers (which has inaugurated, financed, and carried it out) nor those engineers who, by contributing papers and discussions, or by their presence at the meetings, have made this result possible, will ever regret their participation.

PRESIDENT HERMANY.—The Congress will now have the privilege of listening to some remarks from our distinguished guest, Sir William H. White, Pres. Inst. C. E.

SIR WILLIAM H. WHITE.—Mr. President, Ladies and Gentleman: I think the words with which the Secretary concluded his general report err on the side of modesty, and that is the right kind of an error to make, especially when engineering is concerned.

I am to some extent an outsider in this matter, although I do not reckon myself an outsider where the American Society of Civil Engineers is concerned; but looking on as one who has had nothing whatever to do with the organization of this Congress, but who has very large experience in meetings, not merely in Great Britain, but of international character, there and elsewhere, I am happy to say, that neither in conception nor in conduct, nor in organization, nor in the character and scope of the papers printed and the subjects dealt with, and the standing of their authors, nor in the discussions, written and oral, which followed upon the reading of those papers has this Congress any need to fear comparison with any Engineering Congress that has ever been held.

In saying that, I have no intention to make compliments. I am stating that which I believe to be simply the truth, and I heartily congratulate the American Society of Civil Engineers and the Special Committee which undertook the organization of this Congress, and you, Sir, as President of the Congress (turning to President Hermany), for your conduct of its affairs, its meeting at the beginning and now at its close; and last, but not least, to the Secretary of the American Society of Civil Engineers (Applause), upon whom and upon whose staff has necessarily fallen the great burden of the work, both in preparation for, and in carrying out this great scheme. I do most heartily congratulate all concerned upon its unmixed and remarkable success.

I am sure that the volumes of your *Proceedings* when published will be of the greatest value to the Engineering Profession everywhere, and they will mark for all time the standing of the various branches of engineering in this year of 1904; and the fact that out of the total papers contributed about one-half have come from countries other than the United States, I think you will agree with me, will enhance the value of the *Proceedings;* for although it is perfectly true that the enormous extent of this great country and the many problems that it offers, problems of the greatest difficulty and variety, for solution by the engineer—that while the United States, as I say, necessarily shows up in bulk most largely in the proceedings of this Congress, yet you will all agree that in the rest of the world there ought to be found lessons of greatest value and works that are of enormous interest as suggesting to you American engineers possibilities of either varying or enlarging your practice, and which you will consider to be of use.

It is matter of great satisfaction to me, Sir, that on this occasion Great Britain, that is, I should say the British Empire, has been so well represented. In the classification which has been given by the Secretary, we have heard of those who came from England, and Scotland, and Ireland, and Australasia and India; but we know no such subdivision. The British Empire is one and indivisible. British subjects, wherever they live, are British; and, although there are these geographical distinctions, I ask you to consider that at this

Congress the British Empire has been represented in a way that I trust you will think worthy of that intimate connection which we have and always cherish with you on this side of the water. (Applause.)

At the beginning I spoke of the fact that the Institution of Civil Engineers had for the first time in its history made an outing and taken part officially in an International Congress. We did that, as I then said, in order to mark the high distinction in which our Institution holds the American Society of Civil Engineers. We could do no more.

I think I find myself to-day the sole survivor of the British Section of the Congress. I look about, and I think I see no other Englishman here—and we speak of Englishmen as representing the whole British Empire. I think I see no one here; but whether this be so or not, let me at the end say what I said at the beginning, that you have no truer friends and no more interested colleagues than those of us who come here to represent that old institution, the Institution of Civil Engineers. (Applause.)

I have been chiefly engaged under the friendly and kindly guidance of General Craighill in Section G. There seems to be a sort of fate that I should be in Section G. I have been President of Section G in the British Association, and now I find myself a private in this division here. But while my time has been chiefly spent there I have also appeared, as you have heard, in Sections D and H; and as far as my experience goes, there has been an equal interest in all the sections to that which we found to exist in Section G. In Section D we had when I was present a very keen but still friendly discussion on one of these burning subjects that concern the treatment of Steel. Then we passed to Timber and Cement, and I had nothing to say; but I was asked by one gentleman—I think he was concerned with the manufacture of cement—if we did not use large quantities in our ships. I told him we used bitu-mastic cement, when we used it, and we used as little as ever we could. We hope to come to the time when we will use none. I know that does not represent the future of cement in engineering generally, but it represents what we aim at in ship structures, for we want to carry as little as we can of anything that does not contribute to structural strength, and to arrange as best we can to secure protection and durability under these conditions.

Now, if I may say a word as a naval architect, I should say that we who have to build the floating structures that have to sustain stresses arising from conditions that it is impossible to forecast or accurately measure, we look at our brothers, the other civil engineers, who found their structures on the solid earth, with something like envy. If you gentlemen had to float some of the things that you build, they would be very different from what they are. (Laughter and Applause.) And when I look about, as I do often, and see what ample margins are quite reasonably provided, I sometimes wish that the density of sea water was greater than it was made to be.

And then I want to say a word about one allusion which my friend, Mr. Moore, made to modes of transit. It strikes me, you know, that he has perhaps made a claim that cannot be substantiated. He said that the reason they finished so quickly in his Section was that railways were the most rapid mode of transit. I believe that that correctly represents his statement. Now I am going to take exception to that. Do you know that ships are crossing the Atlantic at this date at as nearly as great an average speed as any train crosses the American Continent—and they are going uphill all the time? Not up and down, not with the engine having an easy time when going down a gradient – no, it is collar work all the time; and sometimes when I have been at sea and felt the tremor as the engine forced the vessel through the waves, it has seemed to me as if the ship was almost a sentient thing. Gentlemen, we are all apt to be proud of the particular work with which we are concerned, but I ask you to remember that, if things go well, in about two years there will be afloat upon the Atlantic, ships with which I am personally concerned which are to cross at a mean speed of twenty-nine statute miles per hour. (Applause.) So I say to Mr. Moore, do not let us take the down-hill rate of going, but let us consider the average, and while we admire all that can be done on railroads, do not let us think that transit by water is yet played out, for it is not.

Now, Ladies and Gentlemen, all good things come to an end. It is four weeks to-day since I landed in New York. It seems an age. I have done and seen such things as never before. I have made such multitudes of friends that really sometimes I am ashamed to say I do not recollect them all—not individually, you will understand, going from place to place, being everywhere received with kindness and with such generous hospitality, being treated in so princely a fashion; carried over enormous distances and shown great works in rapid succession; you will understand that one has only a sort of general impression remaining—a general impression of such kindness as can never be forgotten.

Well, my friends are gone—and I stand here alone. (Here Sir William showed evidences of emotion.) I want to say, we shall never forget this visit, we will never forget your kindness. (Applause.)

General Craighill, Past-President, Am. Soc. C. E. – Mr. Chairman, can I say just one word? I am sure that we will never forget Sir William White. (Applause.)

President Hermany: Gentlemen, we will continue our concluding proceedings by calling upon the foreign gentlemen who are members of this Congress to make such addresses on this occasion as they may feel inclined and moved by the occasion to do.

I cannot call them by name, but we will be glad to hear from any foreign member of the Congress who desires to address this assembly.

MR. KARL P. DAHLSTROM.*—Mr. President, Ladies and Gentlemen: During the holding of this International Congress, quite a number of countries have been represented at the general meetings, but as yet there is one country that has not been represented. We are here, a handful of engineers, from a country which has largely contributed to the development of this Louisiana Purchase Territory by sending hundreds and thousands of workers to toil and to break ground. I refer to Sweden. (Applause.)

We are not here as representatives of those workers. We are here representing the leading Swedish Society of Engineers, and we wish to thank the members of the American Society of Civil Engineers, and the Managers, for their kindness in extending an invitation to us.

We have attended the meetings with great interest, I assure you. It is to be regretted that no papers were presented to the Congress originally from our country, for our specialists, for reasons best known to themselves, have omitted to send any papers; but I can assure you that that is not due to any lack of interest on their part. Distance may have had something to do with it, as well as the fact that Sweden is located somewhat out of the way of the main thoroughfares of the commerce of the world; but engineering in Sweden is going on at a lively pace, equal to that in this country. We are developing our manufactures. Everybody knows the high quality of our iron and steel.

We are developing our railroads. We have extended our railroads through the northernmost parts of our country, and you can now travel from the south of Sweden away up to the Midnight Sun in Lapland, at a very comfortable rate.

We have in Sweden something that you have not anywhere else in the world; we have an institution that rewards inventors and scientists, regardless of their nationality, for their achievements for the benefit of mankind. I refer to the Institute Nobel.

As I say, I regret that Swedish experience has not been more largely represented at this Congress, but I hope that as the international work of these Congresses goes on there will be a more intimate intercourse between the Swedish and the American engineering societies. It has been suggested by previous Congresses that there should be formed an international engineering society, and there were people who worked very hard to bring this about, but I suppose the idea, like many bright ideas, was premature. It came too early. Yet it may be realized, and there may be other ideas realized before that, perhaps some international engineering journal, or something to that effect, that might bind the engineers of all nations together.

It may be that perhaps at some future time this International Engi-

* Royal Patent Office, Stockholm, Sweden.

neering Congress may deem it expedient to convene in our beautiful capital in Sweden; and I assure you that if this should happen at any time, or before that time if any members of this American Society of Civil Engineers should find their way to Sweden and wish to study any of our engineering works, just call on your Secretary, Mr. Hunt, for our apdress in Stockholm, and I guess we will find means to put you straight and show you something, and you will be more than cordially welcome.

President Hermany.—A representative from Switzerland is present. Will the gentleman have the kindness to address this meeting?

Mr. Francois Schule, Zurich, Switzerland.—As a member of the International Engineering Congress, I want to thank the American Society of Civil Engineers for organizing this Congress, and to state my belief that there has not been any other great technical congress organized without the support of the State and initiated by a private society such as this; and I must felicitate the Society, its President, and Secretary, for the complete success that has attended their efforts. This Congress will give to every engineer many new ideas with respect to the science of civil engineering. In conclusion, I can only add my tribute to that which was offered by Sir William White in his remarks this morning.

President Hermany.—Will the gentleman from Austria address the meeting?

Mr. O. F. Schonberger, Vienna, Austria.—Mr. Chairman, Ladies and Gentlemen: When I started from Europe to attend this meeting I expected to find a great many of the Austrian engineers at the Congress; but the time was not very convenient for them and therefore I have met only very few here. Since my arrival in America, during these few weeks, I have seen and learned so much and have enjoyed so much hospitality, and formed so many friendships, that I am sure that the Austrian engineers will greatly regret that they did not come in great numbers. In the name of the few who attended I thank the American Society of Civil Engineers for their hospitality, and I hope and believe that when your members come to Austria that the members of the Austrian Society of Engineers and Architects will extend to you the same reception that has been given us here, which has been most cordial and courteous.

President Hermany.—We will now hope to hear from Mr. Bachmetew, of Russia.

Mr. B. Bachmetew, St. Petersburg, Russia.—Mr. Chairman, Ladies and Gentlemen: In several minutes the Congress will be closed and the engineers from all the world who have convened here for about a week will go back to their respective countries. During the past week they have discussed many subjects of great engineering interests. They have not only looked ten years back, but as was well expressed by the chairman of the Mechanical Section, Mr. Haines, they have looked forward.

I am sure that these engineers of all countries who are going to return to their homes will take with them, not only that interest which has come to them as members of the Engineering Congress, but I am quite sure that they will take with them, and will express to their friends, the engineers of their own countries, a sense of the sincerity and great hospitality that they have received here from their American confrères, especially those represented by the American Society of Civil Engineers.

PRESIDENT HERMANY.—May we now hear from the representative from Holland?

MR. R. A. VAN SANDICK.*—Mr. President, Ladies and Gentlemen: As I am the only representative from my country in attendance, I wish to speak a few words in testimony of the services that my country has rendered to the engineering profession in general; for instance, we made our dikes and effected our land reclamation even before America was discovered. Our irrigation and our reclamation societies are as old as the oldest communities in Europe, and that means that we have had an engineering experience of some centuries.

But now I am here in your country, and I have been received by the American engineers with a cordiality that I will never forget, and my only regret is that I have not been present at every session of this International Engineering Congress. The reason that I have not is because I have been admiring your railway system between California and St. Louis, having been detained by a wash-out that held us back four days. On that account it has required more time to make the trip from San Francisco to St. Louis than it would take to go from Holland to New York. (Laughter.)

I want to say that you have quite a different system of building railways in the West than prevails in the East. I believe that you make your railways as simple as is possible; and I think you ought to raise money from your railways, and from the money that you raise you ought to improve them. You have a business-like system of building railways, and the same kind of a business-like system in the building of dikes in California. We, however, in Europe, do not look so much after the business part, at least we are not so money-making. We build our dikes for eternity. (Applause.) And in the same manner that we build our dikes, so we build our railways. I would advise you also to follow our example and to build also for eternity. For your country is new, but it will last for eternity, and therefore you should build your railways and engineering works for eternity.

What I saw in America, what I saw in California, what I have seen in this great World's Fair, and what I have seen of the American Society of Civil Engineers, I had never supposed existed in the world.

* General Secretary of the Royal Institution of Netherlands, Engineers, The Hague, Holland.

When our engineers neglect to visit America I am sure that they know only a portion of the engineering business, and that a very small porion—a very small portion. (Applause.)

Now Ladies and Gentlemen, I want to say that when your people come over to Europe, when your engineers come to Europe, you go, of course, to Germany, you go to England, you go to France; and when you come to Holland you see only the old pictures, you see the evidence of the so-called Golden Century of our People; but I venture to say that we are not yet dead—we are still a live engineering people. (Applause.)

It is true that art is a fine thing, but industry and engineering business, I am proud to say, are alive in Holland, and for that reason when you come to Holland do not forget the Royal Institution of Netherlands Engineers, of which I am the Secretary-General. You can find at my office all sorts of things, and I can advise you, if you want to see very interesting things in a small country.

President Hermany.—We will be pleased to hear from Mr. T. Shima, of New York City, representing Japan.

Mr. T. Shima, New York City.—Mr. President, Ladies and Gentlemen: I have never before been so interested and pleased in attending such a big society of engineers from all parts of the world. I am the engineer of the government railways of Japan and am very much interested in American railroad methods. I have been nearly one year in this country studying along that line, and I have been at this meeting so interested and so happy as never before in my life.

You know my country is small, having only ten thousand square miles of area and it is also narrow.

The railway system is poor, but we have three parallel lines along the island. I think many of the engineers would be interested in visiting my country and inspecting our little system. We have dining cars, sleeping cars and just the same system as in America. We have docks and large harbors. We have water-works and other engineering works that would interest visitors. We are, now, as you know, under difficulties as great as at any time during the twenty-five hundred years since our foundation; but we are trying to push our railway engineering works as energetically as before our difficult war. I wish to express my best wishes for the future prosperity of the American Society of Civil Engineers.

President Hermany.—Our distinguished guest, Sir William White, desires to say a word of adieu. He will do it in his own admirable way.

Sir William H. White.—Ladies and Gentlemen, I must go now. I want to say good-bye. Perhaps in England I may meet many of you again. We did our best to welcome you before. This Congress proves that you valued our feeble attempt, and I hope that you will come again. So I say not "Good-bye," but "Au revoir."

President Hermany.—Are there other gentlemen present who desire to address the meeting? We have not heard from France. Is there not a gentleman from that country who desires to address us? (There was no response.)

President Hermany then continued:

"Ladies and Gentlemen, at the close of this session of the General International Engineering Congress, I would say that, judging from the character of the addresses we have listened to this morning, one might reasonably have some doubt as to whether this is an opening or a closing session. Judging from the extraordinary interest manifested and the number of persons present, the same interest is unmistakably evident now as upon the first morning of this week when we assembled here. I am constrained to say, after having looked into the countenances of the people then, and now again looking into your countenances, I see a depth of interest that bespeaks a devotion to the cause or profession of engineering that I may say is evidenced by not a single passive brain in this assembly.

There is not a countenance here that exhibits the slightest degree of inertness. This it has never been my privilege to have witnessed before, as I do upon this occasion, and as was manifest upon the first morning of our assembly.

In closing this Second International Congress held on American soil there remains but little to say. Much has been said to the point. The success—the eminent success, which it has attained is a matter for rejoicing and congratulation of us all.

It is perhaps a little bold to say it, but from my viewpoint and from my intercourse with members attending this Congress, I feel justified in saying, that when its work shall have been compiled and published it will establish the fact that the work accomplished here will not rank second to that of any preceding International Engineering Congress.

I, therefore, heartily congratulate you upon the work that has been done here. As was said in the opening address on Monday, the acquisition of the Louisiana Territory by peaceful diplomacy finds a most remarkable expression in this grand exposition, the centennial commemoration of that event—the grandest centennial the world has ever had.

The acquisition of this territory formed an epoch in our national history. Is it venturing too far to say that this International Engineering Congress will prove to be an epoch in American engineering?

It is now time to speak the final word. With kindest regrets and warmest friendship, with the greatest good will to you all, I now, in the name of the American Society of Civil Engineers, adjourn this General International Congress. (Applause.)

The assemblage was now about to disperse, but was invited to remain a few moments, by Secretary Hunt, in order to hear an announcement from Mr. Corthell.

Mr. E. L. Corthell.—Gentlemen, I asked the Secretary to do this, but he thought that I ought to make the explanation; at the request of the Chairman of the Board of United States Representatives to the International Navigation Congress, General Raymond, I wish to call your attention to the fact that next September, 1905, there will be held in Milan, Italy, an International Navigation Congress.

I have sent to every member of the American Society of Civil Engineers and also to about one hundred Associate Members the Propaganda, as we call it, of our Board. The Secretary of the Milan Congress has sent me from Milan about 200 of the programmes of that Congress in English, and if any member interested in that International Navigation Congress will write to me at No. 1 Nassau Street, New York City, I have about 150 of these programmes left, which I will be very glad to send to anyone.

Thereupon the International Engineering Congress adjourned *sine die.*

INTERNATIONAL ENGINEERING CONGRESS.

The following correspondence was presented to the Board of Direction of the American Society of Civil Engineers, November 1st, 1904, and is here printed by special order of the Board.

"INTERNATIONAL ENGINEERING CONGRESS,
"COMMITTEE IN CHARGE.
"ST. LOUIS, MO., October 7th, 1904.

"MR. CHARLES WARREN HUNT,
"*Sec., Int. Engineering Congress.*

"MY DEAR SIR:—

"It is with great pleasure that I enclose a copy of the resolution this day unanimously adopted by the Committee in Charge, and I trust that I may be permitted to add, from my personal acquaintance with the matter, that it is to your unflagging zeal and executive ability that the acknowledged success of the International Engineering Congress is mainly due.

"Very sincerely yours,
"H. S. HAINES,
"*Chairman.*"

"INTERNATIONAL ENGINEERING CONGRESS,
"COMMITTEE IN CHARGE.
"ST. LOUIS, MO., October 7th, 1904.

"MR. CHARLES WARREN HUNT,
"*Sec., American Society of Civil Engineers.*

"DEAR SIR:—

"At a meeting of the Committee in Charge of the International Engineering Congress held this day at the Administration Building, Louisiana Purchase Exposition, the following resolutions were unanimously adopted:

"*Resolved,* That it is the sense of the Committee in Charge of the International Engineering Congress that warm thanks should be given to our Secretary, Mr. Chas. Warren Hunt, for the successful outcome which is mainly due to his ability in the arrangement and execution of the manifold details of preparation for the Congress and for the unusual labor imposed by the performance of this important extra and special duty.

"*Resolved,* That a copy of this resolution be furnished to the Board of Direction of the American Society of Civil Engineers, with the hope that the membership of the Society in general may be appraised of the value of Mr. Hunt's services in this connection.

"Very truly yours,
"H. S. HAINES,
"*Chairman.*"

ANNOUNCEMENTS.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

MEETINGS.

Wednesday, December 7th, 1904.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "Probable Wind Pressure Involved in the Wreck of the High Bridge over the Mississippi River, on Smith Avenue, St. Paul, Minn., August 20th, 1904," by C. A. P. Turner, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings*.

Wednesday, December 21st, 1904.—At this meeting a paper, entitled "The Reclamation of River Deltas," by J. Francis Le Baron, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings*.

UNIVERSAL EXPOSITION, ST. LOUIS, 1904.

The Society has undertaken to provide for an engineering exhibit and the establishment of Headquarters for visiting engineers **in the center of the Liberal Arts Building,** and the Board of Direction has appropriated sufficient funds to defray the necessary expense.

This matter is in the hands of the following committee:

Robert Moore, M. Am. Soc. C. E., St. Louis, Mo., *Chairman*.
Edward C. Carter, M. Am. Soc. C. E., Chicago, Ill.
Mordecai T. Endicott, M. Am. Soc. C. E., Washington, D. C.
James L. Frazier, " " Frankfort, Ind.
William Jackson, " " Boston, Mass.
Emil Kuichling, " " New York, N. Y.
J. L. Van Ornum, " " St. Louis, Mo.
John F. Wallace, " " Chicago, Ill.
O. E. Mogensen, *Sec'y*, " " St. Louis, Mo.

PRIVILEGES OF LOCAL SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

The **Boston Society of Civil Engineers** will welcome any member of the American Society of Civil Engineers at its library and reading room, 517 Tremont Temple, Boston, which is open on week days from 9 A. M. to 5 P. M. Members will also be welcome at the meetings, which are held in the same building, on the evenings of the fourth

Wednesday in January, and the third Wednesdays of other months, except July and August.

The rooms of the **St. Louis Engineers' Club,** in the business center of St. Louis, will be kept open during the World's Fair season, May 1st to December 1st, 1904, and visiting engineers are cordially invited to use them for mail, telephone service, information, etc.

The courtesies of the **Engineers' Society of Western Pennsylvania** have been extended to members of the American Society of Civil Engineers. The rooms of the Society, 410 Penn Ave., Pittsburg, Pa., are open at all times, and meetings are held as follows, except during July and August. Regular Section, Third Tuesdays; Chemical Section, Thursdays following third Tuesdays; Mechanical Section, first Tuesdays; Structural Section, Fourth Tuesdays.

The Western Society of Engineers, Monadnock Block, Chicago, Ill., tenders to members of this Society the use of its rooms and facilities, together with the good offices of its Secretary and of a special committee appointed for that purpose.

The Civil Engineers' Club of Cleveland, Ohio, invites members of this Society to make use of the Club rooms, at any time when in Cleveland. Cards will be furnished on application to the Secretary, Mr. J. C. Beardsley.

SEARCHES IN THE LIBRARY.

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many such searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling, compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

Copies of all lists of references are filed, so that in many cases it is only necessary to make a typewritten copy, which reduces the cost of searches to a minimum.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

ACCESSIONS TO THE LIBRARY.

October 12th to November 7th, 1904.

DONATIONS.*

BRITISH SEWAGE WORKS.

And Notes on the Sewage Farms of Paris and on Two German Works. By M. N. Baker. Cloth, 9 x 6 in., 146 pp. New York, The Engineering News Publishing Co., 1904. $2.

The introduction says: In the early history of American works for the treatment of sewage, *Engineering News* commissioned the writer to prepare a series of articles on "Sewage Purification in America." Some ten years having elapsed, during which many new, or at least modifications of old, methods had come to the front, particularly in Great Britain, the writer seized an opportunity, afforded by *Engineering News*, to visit representative British sewage works, and to meet leading British experts in sewage disposal. Twenty-four British sewage works were visited. They include examples of both old and new methods of treatment and of large and small works. Aside from five sewage farms and three chemical precipitation plants, the works combine two or more methods of treatment and are not easily classified. The readiest and perhaps the most reasonable basis of classification for the sixteen combined works appears to be the final method of treatment employed. Using this, we have eleven works with contact beds and five with percolating filters. The Contents of the book are: Part I, Works Employing Contact Beds for Final Treatment.—Part II, Works Employing Percolating Filters for Final Treatment.—Part III, Sewage Farms.—Part IV, Chemical Precipitation Works. Appendix I contains descriptions of German and French plants, and Appendix II, definitions and descriptions of sewage treatment for the convenience of any readers not familiar with the processes now in use. In the index the places are grouped alphabetically under the processes.

EARTH DAMS: A STUDY.

By Burr Bassell, M. Am. Soc. C. E. Cloth, 9 x 6 in., 70 pp. New York, Engineering News Publishing Company, 1904. $1. (Donated by the author.)

The author states that the literature upon the subject of earth dams is voluminous, but much of it is inaccessible and far from satisfactory. No attempt will be made here to collate this literature, or to give a history of the construction of earth dams; the object will rather be to present such a study as will make clear the application of the principles underlying the proper design and erection of this class of structures. Appendix II contains a bibliography on the subject; and there is an index of two pages.

UNTECHNICAL ADDRESSES ON TECHNICAL SUBJECTS.

By James Douglas. Cloth, 7 x 5 in., 84 pp. New York, John Wiley & Sons, 1904. $1.

These addresses, by a former President of the American Institute of Mining Engineers, were given on different occasions, such as the California meeting of the Institute, September, 1899; before the School of Mines and Metallurgy of the University of Missouri, May 30th, 1901; and before the Michigan College School of Mines, April 22d, 1904. In reprinting them, omissions are made occasionally to avoid repetition. In other cases, in order to bring the information up to date or to amplify the subject, considerable additions are interpolated; but these are enclosed in brackets. The Contents are: The Characteristics and Conditions of the Technical Progress of the Nineteenth Century; The Development of American Mining and Metallurgy, and the Equipments of a Training School; Wastes in Mining and Metallurgy.

CEMENTS, MORTARS AND CONCRETES: THEIR PHYSICAL PROPERTIES.

By Myron S. Falk, Jun. Am. Soc. C. E. Cloth, 9 x 6 in., 6 + 176 pp. New York, M. C. Clark, 1904. $2.50. (Donated by the author.)

The preface states that the purpose of this treatise has been to set forth as concisely as possible the physical properties of cement and cement mixtures, with principal reference to those properties which concern the engineer. The results of investigations made upon these materials have been examined with great care, and it has been the author's object to abstract, classify and summarize all the reliable data extant, filling in certain gaps with data of his own. The following headings outline, for the greater

* Unless otherwise specified, books in this list have been donated by the publishers.

part, the scope of the work: General Physical Properties: Changes in Volume when Setting; Coefficient of Expansion Due to Temperature Changes; The Action of Sea Water and Salt; Porosity and Permeability; Effect of Freezing; Adhesion of Iron Rods to Cement Mixtures; Fatigue of Cement Mixtures; General Elastic Properties: Tensile and Compressive Properties; Coefficient of Elasticity; Elastic Limit; Ultimate Resistance; Flexure Properties; Coefficient of Elasticity; Modulus of Rupture; Shearing Resistance. A report of a special committee of the American Society of Civil Engineers on uniform tests of cement is given in Appendix I, and the Constitution of Portland Cement, by Clifford Richardson, in Appendix II. There is an index of subjects and one of authors, together covering six pages.

THE FIELD PRACTICE OF RAILWAY LOCATION.

By Willard Beahan, M. Am. Soc. C. E. Cloth, 9 x 6 in., 252 pp. New York, Engineering News Publishing Company, 1904.

It is stated in the preface that the object of this book is to record the methods commonly used by American engineers in the West in the location of railroads built since the war. The writer is not aware of any similar book on this subject. The book is primarily for chiefs of party, for engineers obliged to make their first location, and for students. As much of economics, traffic, transportation, topography, geology and the locomotive is embraced as those who use it will probably have time to study or consider. This is a book for the instruction of the inexperienced in this kind of field work, and it is made as plain and as consecutive in the statement of practical routine as possible. No attempt has been made to exhaust the economic phases of the subject. In so far as it is possible, each chapter contains all the information needed for that part of the work. The economic units and the principles established by the late A. M. Wellington in his valuable book, "The Economic Theory of the Location of Railways," are here used and referred to by chapter and section in each case. There is an index of two pages.

REINFORCED CONCRETE.

Part I, Methods of Calculation. By A. W. Buel. Part II, Representative Structures. Part III, Methods of Construction. By C. S. Hill. Cloth, 9 x 6 in., 434 pp. New York, Engineering News Publishing Company, 1904. $5.00.

It is stated in the preface that in preparing this book the authors have had in mind a treatise for designing and constructing engineers following American practice and governed by the conditions which prevail in America. Theoretical discussions have been omitted, and in their place have been supplied practical working formulas, examples of representative structures, and records of actual practice in the selection of materials and of methods of workmanship and construction. For convenience of classification the book is divided into three parts or sections. In Part I are given working formulas for the calculation of all classes of structures in reinforced concrete and such facts about the properties of concrete and steel as are necessary in developing economical engineering designs. Part II contains illustrations and descriptions of a large number of representative structures of reinforced concrete. These record actual practice in design and show the adaptability of the new material to various types and forms of structures. Materials, workmanship and methods of construction are considered in Part III and are illustrated by numerous examples from actual practice. In this part especial attention is given to the construction of centers and forms for concrete work and to methods of facing and finishing exposed concrete surfaces. There is an index of six pages.

NOTES ON HYDROLOGY;

And the Application of its Laws to the Problems of Hydraulic Engineering. By Daniel W. Mead, M. Am. Soc. C. E. Cloth, 9 x 6 in., 202 pp. Chicago, 1904. (Donated by the author.)

These notes are intended by the author to form the basis for an introductory study of the fundamental phenomena of Hydrology, on which the applied science of Hydraulic Engineering should be based. The volume of literature covering many of the various branches of this subject is very great. Unfortunately, however, there is no single treatise which discusses the entire subject, and which can be utilized as a textbook, or reference book, to which the student may turn when investigating the various branches of this science. The lack of such a work is the reason for the preparation of these notes, which are intended to be used in connection with various publications to which references are given. The principles and laws of Hydrology must, of necessity, be based almost entirely on extended and long-continued observations, consequently the writer has utilized the observations available from a great many sources, and for long periods of time.

THE BUILDING ESTIMATOR.

By William Arthur. Cloth, 6 x 4 in., 150 pp. Omaha, Neb. (William Arthur), 1904. $1.50. (Donated by the author.)

The first part of the book gives approximate estimating for architects, engineers, etc., and methods showing how to obtain the value of buildings. Several pages of percentages are also given for residences, warehouses, stores and flats, schools, manufacturing and business buildings. The percentage of excavation, mason work, lumber, mill work, glass, carpenter labor, paint, hardware, tin, slate, plumbing, etc., is given under each class, and from six to twelve examples are used from actual bids put in or work done. There is a two-paged index.

CARBURATION ET COMBUSTION DANS LES MOTEURS A ALCOOL.

Par E Sorel. Paper, 9 x 5 in., 280 pp. Paris, Vve. Ch. Dunod, 1904. Paper, 8 francs; boards, 9 francs 50.

Much has been said for and against the using of alcohol in motors. Its partisans declare that it can be substituted for light forms of petroleum in any kind of a motor, without a disagreeable odor or smoke being perceptible. Its opponents accuse it of producing acids attacking the cylinders and suction-valves. The author says that these eulogies and reproaches are without foundation; all depends upon the circumstances of using and the manner of mingling the air and the combustible. In this new work the author aims to show the conditions that are necessary and sufficient.

ETUDE THEORIQUE ET PRATIQUE SUR LA VAPORISATION.

Méthode pour Augmenter Considérablement le Rendement des Générateurs à Vapeur. Par M. E. Wickersheimer. Paper, 10 x 6 in., 76 pp. Paris, Vve. Ch. Dunod, 1904. 3 francs 50.

The author has attempted, in this study, to show a new method for considerably in creasing the efficiency of boilers. In this way he aims to economise a large proportion of the fuel without hurting the metal in the least with the heat and even preserving it from harm. The first part of the work consists in theoretical considerations and in laboratory practice; the second relates to industrial experience made in order to verify the Bez method.

ETUDE THEORIQUE DES ALLIAGES METALLIQUES.

Par Léon Guillet. Paper, 10 x 6 in., 232 pp. Paris, Vve. Ch. Eunod, 1904. 7 francs 50.

It is stated in the preface that recent researches on alloys have shown that their physical, chemical and mechanical properties depend upon the state in which the different metals entering into their composition are found. The author's aim is to study the different methods by which this knowledge is acquired; to show what has been already accomplished; and to write of the different alloys used in manufacturing. This volume is devoted essentially to theory; the author has repeated here the law on the phases of which the theory of alloys undoubtedly depend; in the following chapters he has studied all methods leading to the clearing up of metallurgical products. Each chapter has three divisions: 1st, the principle; 2d, the methods; 3d, the examples. The examples are taken from important work connected with the names of MM. Le Chatelier, Osmond and Hadfield.

ROCK EXCAVATION.

Methods and Cost. By Halbert Powers Gillette, M. Am. Soc. C. E. Cloth, 5 x 7 in., 8 + 376 pp. New York, M. C. Clark, 1904. $3.00 (Donated by the author.)

The author says that, while the scope of the book is wider than at first sight appears desirable, since it includes quarrying, open-cut excavation, trenching, subaqueous excavation, tunneling and under-ground excavation, still it should be remembered that the main elements of cost are of much the same quality, and that the differences are principally those of quantity. Thus it may require eight feet of drill hole per cubic yard of tunnel excavation as compared with half a foot for open cut work. The cost of drilling per cubic yard is obviously much greater in the tunnel than in the open cut, but the methods of drilling and the cost per foot of drill hole may be practically indentical. There is indeed much in common in all classes of rock excavation in spite of many detailed differences. The Chapter Headings are: Rocks and Their Properties; Methods and Cost of Hand Drilling; Machine Drills and Their Use; Steam and Compressed Air Plants; The Cost of Machine Drilling; Cost of Diamond Drilling; Explosives; Charging and Firing; Methods of Blasting; Cost of Loading and Transporting Rocks; Quarrying Stone; Open Cut Excavation; Methods and Costs on the Chicago Drainage Canal; Cost of Trenches and Subways; Subaqueous Excavation; Cost of Railway Tunnels; Cost of Drifting; Shaft Sinking and Stoping. The book contains an index of six pages.

Gifts have also been received from the following:

Am. Ry. Master Mechanics' Assoc. 1 bd. vol.
Atlanta & West Point R. R. Co. 2 pams.
Brit. Fire Prevention Comm. 1 pam.
Buffalo & Susquehanna R. R. Co. 1 pam.
Chicago, Burlington & Quincy R. R. Co. 1 pam.
Chicago, Rock Island & Pacific Ry. Co. 1 pam.
City Record. 6 bd. vol.
Cohen, Mendes. 67 pam.
Davis, J. 1 pam.
Dunod, Vve. Ch. 1 vol.
Eno, F. H. 1 vol.
Fisk, W. L. 1 pam.
Georgia Southern & Florida Ry. Co. 1 pam.
Gloucester, Mass.—City Council. 1 bd. vol.
Great Britain—Patent Office. 7 pam., 5 vol.
Great Northern Ry. Co. 1 pam.
Imperial Earthquake Investigation Comm. 1 pam.
Inst. of Civ. Engrs. 1 vol.
Inst. of Marine Engrs. 1 bd. vol.
Iowa Eng. Soc. 1 pam.
Königliche Technische Hochschule zu Danzig. 1 pam.
Louisville & Nashville R. R. Co. 1 pam.
McGill Univ. 1 bd. vol.
Madras, India—Pub. Works Dept. 1 pam.
Mass.—Statistics of Labor, Bureau of. 2 bd. vol.
Mead, D. W. 1 bd. vol.
Mich.—Health, Board of. 1 bd. vol.
Milwaukee—Board of Pub. Works. 1 vol.
Miss. Wire Glass Co. 1 bd. vol.
Montreal—City Council. 1 bd. vol.
New York—Dept. of Health. 3 bd. vol.
New York—State Engr. and Surv. 2 bd. vol.
New York Central & Hudson River R. R. Co. 1 pam.
Nichols, O. F. 1 pam.
North, E. P. 2 bd. vol.
North-East Coast Inst. Engrs. & Ship-builders. 1 bd. vol.
Northern Pacific Ry. Co. 1 pam.
Queensland—Commr. for Rys. 1 vol.
Reading—Board of Water Commrs. 1 bd. vol.
Reading Co, 1 pam.
Ricketts, P. C. 1 vol.
Soc. Belge des Ingénieurs et des Industriels. 1 pam.
Soc. of Arts. 1 pam.
South Australia.—Rys. Commr. 1 pam.
Stanley Electric Mfg. Co. 1 map.
Toledo, St. Louis & Western R. R. Co. 1 pam.
Toronto—Provincial Board of Health. 1 pam.
Union Pacific R. R. Co. 1 pam.
U. S. Bureau of Forestry. 5 pam.
U. S. Bureau of Statistics. 1 vol.
U. S. Div. of Foreign Markets. 1 pam.
U. S. Navy Dept. 1 vol.
U. S. Office of Exper. Stations. 3 pams.
U. S. War Dept. 18 Specif.
U. S. Weather Bureau. 1 pam.
Univ. of Ill. 1 pam.
Wabash R. R. Co. 1 pam.
Victoria—Melbourne and Met. Board of Works Engr.-in-Chf. 1 pam.
Western Ry. of Ala. 1 pam.
Wis. Central Ry. Co. 1 pam.

BY PURCHASE.

A Text Book on Static Electricity. By Hobart Mason. New York, McGraw Publishing Company, 1904.

Refuse Disposal and Power Production. By W. Fancis Goodrich. New York, E. P. Dutton.

Scientific American Reference Book. Compiled by Albert A. Hopkins and A. Russell Bond. New York, Munn & Company, 1905.

SUMMARY OF ACCESSIONS.

October 12th to November 7th, 1904.

Donations (including 29 duplicates)	188
By purchase	3
Total	191

MEMBERSHIP.

ADDITIONS.

MEMBERS.

Members		Date of Membership.
ALMIRALL, RAYMOND FRANCIS. Archt., 51 Chambers St., New York City		Oct. 5, 1904
BISHOP, HUBERT KEENEY. Supt., Public Works, Hudson, N. Y.		Oct. 5, 1904
BLACKWELL, FRANCIS OGDEN. Cons. Engr., 49 Wall St., New York City		Oct. 5, 1904
CARTER, WILLIAM J. City Engr. (Res., 2042 Woodland Hills Ave.), Cleveland, Ohio	Jun. Assoc. M. M.	Feb. 5, 1895 Sept. 7, 1898 Nov. 1, 1904
CHAMBERLIN, CHESTER HARVEY. Div. Engr., Tex. & Pac. Ry., Boyce, La		Oct. 5, 1904
CHAPPELL, THOMAS FENNING. 45 Broadway, New York City.		Oct. 5, 1904
EASTWOOD, JOHN THOMPSON. First Asst. Engr., Sewerage, Sewerage and Water Board, 602 Carondelet St., New Orleans, La.	Jun. Assoc. M. M.	Mar. 6, 1894 Feb. 1, 1899 Sept. 6, 1904
FORD, WILLIAM GRIFFING. 200 Montague St., Brooklyn, N. Y.		Oct. 5, 1904
FORGIE, JAMES. Chf. Asst. Engr., North River Div., P. R. R. Tunnels, 1 West 34th St., New York City		Oct. 5, 1904
FORT, EDWIN JOHN. Dept. of Highways, Municipal Bldg., Brooklyn, N. Y.	Assoc. M. M.	April 1, 1896 Nov. 1, 1904
FYFE, JAMES LINCOLN. 1106 The Rookery, Chicago, Ill.		Oct. 5, 1904
HANNA, JOHN HUNTER. Asst. Chf. Engr. and Supt., Capital Traction Co., 36th and M Sts., Washington, D. C.	Assoc. M. M.	April 3, 1901 Nov. 1, 1904
HARDY, EDWARD DANA. 3107 Eleventh St., N. W., Washington, D. C.		Oct. 5, 1904
HASSKARL, JOSEPH FREDERICK. Supt. of Constr., U. S. Engr. Dept. (Res., 1603 West Girard Ave.), Philadelphia, Pa.	Assoc. M. M.	Mar. 5, 1902 Nov. 1, 1904
HOHL, LEONHARD JOHN. Mgr., Cherokee Gold Dredging Co., Oroville, Butte Co., Cal.		Oct. 5, 1904
HUNT, RUFUS CAMERON. 718 St. Nicholas Ave., New York City		Oct. 5, 1904
JOHNSON, LEWIS JEROME. Asst. Prof. of Civ. Eng., Harvard Univ., 309 Pierce Hall, Cambridge, Mass.	Assoc. M. M.	Sept. 4, 1901 Nov. 1, 1904
KOWER, HERMANN. Asst. Prof. of Drawing, Univ. of California, Berkeley, Cal.		Oct. 5, 1904
LAFORGE, FREDERICK WILLIAM. Junior Engr., U. S. Engr. Dept., Ft. Terry, N. Y.		Oct. 5, 1904
LANG, OTTO HEINRICH. With Tex. & Pac. Ry. Co., Dallas, Tex.		Oct. 5, 1904
LARRABEE, WILLIAM DOMINICK. Supt. and Chf. Engr., Los Angeles Pacific R. R. Co., 316 West 4th St., Los Angeles, Cal.		Oct. 5, 1904

		Date of Membership.
Miller, Frank. Chf. Engr., Snare & Triest Co., 39 Cortlandt St., New York City		Oct. 5, 1904
Moir, Ernest William. Director, S. Pearson & Son, Ltd., 10 Victoria St., Westminster, S. W., London, England.		Sept. 7, 1904
Molera, Eusebius Joseph. 2025 Sacramento St., San Francisco, Cal.		Oct. 5, 1904
Neilson, George Harrison. Gen. Mgr., Braeburn Steel Co., Braeburn, Pa.		Oct. 5, 1904
Pope, Willard. Vice-Pres. and Engr., Canadian Bridge Co., Ltd., Walkerville, Ont., Canada		Oct. 5, 1904
Reed, Wendell Monroe. Dist. Engr., U. S. Geological Survey, Reclamation Service, Roswell, N. Mex.		Oct. 5, 1904
Ricketts, Louis Davidson. Globe, Ariz.		Oct. 5, 1904
Sears, Walter Herbert. Div. Engr., Croton River Div., N. Y. Aqueduct Comm., Katonah, N. Y.		Oct. 5, 1904
Seurot, Paul Albert. Office Engr., North River Div., Penn., N. Y. & L. I. R. R. Co., 1 West 34th St., New York City.	Jun.	April 30, 1895
	Assoc. M.	Oct. 4, 1899
	M.	Nov. 1, 1904
Strickler, Gratz Brown. Gen. Supt., Richardson & Burgess, Inc., 613 Colorado Bldg., Washington, D. C.	Jun.	Jan. 5, 1897
	Assoc. M.	Jan. 4, 1899
	M.	Nov. 1, 1904
Talcott, Harry Randolph. Asst. Engr., Constr. Dept., B. & O. R. R. Co., Room 8, Walsh Bldg., Cumberland, Md.		Oct. 5, 1904
Teichman, Frank. Care, U. S. Geological Survey, Livingstone, Ariz.		Oct. 5, 1904
Walker, James Wilson Grimes. Civ. Engr., U. S. N., Navy Yard, Charleston, S. C.		Oct. 5, 1904
Woodard, Silas H. Care, Penn., N. Y. & L. I. R. R., 345 East 33d St., New York City.	Assoc. M.	Jan. 2, 1901
	M.	Nov. 1, 1904

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Bamford, William Brokaw. Archt. and Civ. Engr., 1123 Broadway, New York City.	Jun.	June 2, 1903
	Assoc. M.	Oct. 5, 1904
Bowler, Frank Colburn. Millinocket, Me.		Oct. 5, 1904
Brown, Stephen Pearson. Cons. Engr., Stratford, Conn.		Oct. 5, 1904
Carroll, Charles Joseph. Div. Engr., Mex. International R. R., Durango, Mexico		Oct. 5, 1904
Clark, Roscoe Nathaniel. 800 Main St., Hartford, Conn.		May 4, 1904
Hawxhurst, Robert, Jr. Chf. Engr., Kohala & Hilo Ry. Co., Hilo, Hawaii		Sept. 7, 1904
Johnston, John Parry. 822 Marquette Bldg., Chicago, Ill.	Assoc.	April 3, 1894
	Assoc. M.	Nov. 1, 1904
Kieffer, Stephen Ephraim. 2515 Dwight Way, Berkeley, Cal.		Oct. 5, 1904
Macdonald, Archibald Angus. 111 Fifth Ave., New York City	Jun.	June 4, 1901
	Assoc. M.	Oct. 5, 1904

		Date of Membership.
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	Assoc. M.	Oct. 5, 1904
RUNYON, WILLIAM KEEPER. 111 Central Ave., Newark, N. J.	Jun.	April 5, 1898
	Assoc. M.	Oct. 5, 1904
SHERTZER, TYRRELL BRADBURY. Asst. Engr., Rapid Transit R. R. Comm., 621 West 152d St., New York City		Oct. 5, 1904
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THORP, RICHARD FENWICK. Munaar, Madura Dist., South India		Oct. 5, 1904
TIERNAN, AUSTIN KING. Box 441, Salt Lake City, Utah		Oct. 5, 1904
WACHTER, CHARLES LUCAS. Constr. Engr., Cableway Dept., Lidgerwood Mfg. Co., 96 Liberty St., New York City	Jun.	June 3, 1903
	Assoc. M.	Oct. 5, 1904

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CAIRD, JAMES MORTON. 271 River St., Troy, N. Y.	Oct. 5, 1904

JUNIORS.

BRILLHART, JACOB HERBST. Box 42, Port Griffith, Luzerne Co., Pa.	Nov. 1, 1904
HUTCHINGS, JOHN BACON, Jr. 217 Waldron St., West Lafayette, Ind.	Sept. 6, 1904

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SAFFORD, EDWARD STANLEY............Civ. and Cons. Engr., 26 Cortlandt St., Room 215, New York City.
SIMPSON, GEORGE FREDERIC.............Asst. Engr., Rapid Transit R. R. Comm., 318 West 135th St., New York City.
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SWIFT, WILLIAM EVERETT.............U. S. Reclamation Service, Chamber of Commerce Bldg., Denver, Colo.

THOMPSON, MACKEY JAMES............901 Pioneer Press Bldg., St. Paul, Minn.

TIDD, ARTHUR WARREN...............New York Aqueduct Comm., Pawling, N. Y.

WILSON, PERCY HARTSHORNE...........Secy. and Treas., River & Harbor Impvt. Co., 501 Mariner & Merchant Bldg., Philadelphia, Pa.

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JUNIORS.

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BRISLEY, EDWARD BETTS.............Crocker-Wheeler Co., 607 Empire Bldg., Pittsburg, Pa.

FOSS, JOHN HARRISON.................Engr., Hamakua Ditch, Paia, Maui, Hawaii.

GEORGE, HUGH MELVIN...............Laurel, Miss.

HIGGINSON, JONATHAN YATES..........Saugatuck, Conn.

HORTON, SANDFORD...................54 West 129th St., New York City.

MILLER, STANLEY ALFRED.............Asst. Engr., S. Pearson & Son, Harbour Works, Coatzacoalcos, V. C., Mexico.

PLOGSTED, WALTER JOHN.............Care, Sanderson & Porter, 323 Baronne St., New Orleans, La.

WELLER, FRANCIS REPETTI............Associated with Henry A. Pressey, Civ. and Hydr. Engr., 408 Colorado Bldg., Washington, D. C.

WHITNEY, THOMAS BRYAN, Jr..........19 West 38th St., New York City.

WHITSON, ABRAHAM UNDERHILL........Aspinwall, Pa.

DEATHS.

BOUSCAREN, LOUIS FREDERIC GUSTAVE..Elected Member, April 7th, 1875; died November 6th, 1904.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(October 11th to November 5th, 1904.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Engrs. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Engrs., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 25c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c.
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill., 10c.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England, 25c.
(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Engineer*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, $1.
(29) *Journal*, Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des École Spéciales de Gand*, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Économique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill., 10c.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Engrs., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
(47) *Mechanical Engineer*, Manchester, England.
(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.
(57) *Colliery Guardian*, London, England.
(58) *Proceedings*, Eng. Soc. W. Pa., 410 Penn Ave., Pittsburg, Pa., 50c.
(59) *Transactions*, Mining Inst. of Scotland, London and Newcastle-upon-Tyne, England.
(60) *Municipal Engineering*, Indianapolis, Ind., 25c.
(61) *Proceedings*, Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E. London, England.
(64) *Power*, New York City, 20c
(65) *Official Proceedings*, New York Railroad Club, Brooklyn, N. Y., 15c.
(66) *Journal of Gas Lighting*, London, England, 15c.
(67) *Cement and Engineering News*, Chicago, Ill., 25c.
(68) *Mining Journal*, London, England.
(69) *Mill Owners*, New York City, 10c.
(70) *Engineering Review*, New York City, 10c.
(71) *Journal*, Iron and Steel Inst., London, England.
(72) *Street Railway Review*, Chicago, 30c.
(73) *Electrician*, London, England, 18c.
(74) *Transactions*, Inst. of Min. and Metal., London, England.
(75) *Proceedings*, Inst. of Mech. Engrs., London, England.
(76) *Brick*, Chicago, 10c.
(77) *Journal*, Inst. Elec. Engrs., London, England.

LIST OF ARTICLES.

Bridge.

Erection of the North-West Miramichi Bridge, Newcastle, N. B.* H. D. Bush. (5) v. 17, Pt. 1.
The Comparative Economy of Various Types of Highway Bridges. C. B. Wing. (Paper read before the Technical Soc. of the Pacific Coast.) (1) Sept.
Skew Bridge of Armoured Concrete.* (12) Oct. 7.
A Special Floor System for a Deck Bridge.* (14) Oct. 15.
The Ticonic Foot Bridge, Waterville, Me.* (14) Oct. 15.
A New Graphical Method for Stresses in Three-Hinged Arches.* J. W. Balet. (13) Oct. 20.
Travelers for Steel Spans. (Committee Rept. to the Assoc of Ry. Supts. of Bridges and Buildings.) (40) Oct. 21; (13) Oct. 27; (15) Oct. 28.
A Highway Bridge Floorbeam Connection.* (14) Oct. 22.
The Care of Railroad Trestles While Being Filled. (Committee Rept. to the Assoc. of Ry. Supts. of Bridges and Buildings.) (14) Oct. 22; (13) Oct. 27; (15) Oct. 28; Abstract (40) Oct. 21.
Forest Park Bridge of the Wabash at St. Louis.* A. O. Cunningham, M. Am. Soc. C. E. (15) Oct. 28; (14) Nov. 5.
Concrete Bridges on the St. Louis & San Francisco Ry.* (13) Nov. 3.
The Bellows Falls Highway Bridge.* (14) Nov. 5.
Construction Economique des Larges Ponts en Pierre: Deux Ponts Minces à Chaque Tête Portant un Plancher en Béton Armé: Ponts de Luxembourg et de Toulouse.* M. Séjourné. (38) Oct.
Pièces en Treillis Chargées de Bout, Brides Parallèles et non Parallèles. A. Vierendeel. (30) Oct.
Nouvelle Détermination Géométrique des Efforts dans les Poutres Droites à Travées Continues.* Farid Boulad. (33) Oct. 8.
Le Pont Suspendu Élisabeth, à Budapest.* A. Bidault des Chaumes. (33) Oct. 15.

Electrical.

A Modern Machine: The Direct Current Motor.* F. S. Keith. (5) v. 17, Pt. 1.
The Laying of the Commercial Pacific Cable. Frank P. Medina. (Paper read before the Technical Soc. of the Pacific Coast.) (1) Sept.
Localisation of Faults on Low-Tension Networks.* W. E. Groves, A. M. I. E. E. (77) Sept.
Some Properties of Alternators under Various Conditions of Load.* A. F. T. Atchison, Assoc. I. E. E. (77) Sept.
Eddy Currents in Solid and Laminated Masses.* M. B. Field, M. I. E. E. (77) Sept.
Armature Reactions in Alternators, with some Notes on the Running of Synchronous Motors.* H. W. Taylor. (77) Sept.
Alternating-Current Commutator Motors.* F. Creedy. (77) Sept.
The Telautograph.* James Dixon. (42) Oct.
The Distribution of Magnetic Induction in Multipolar Armatures.* W. M. Thornton. (26) Oct. 7.
Simultaneous Telegraphy and Telephony. Joseph Hollos. (Paper read before the International Elec. Cong.) (73) Oct. 7.
Effects of Continuous and Alternating Currents on the Dielectric Strength of Insulators.* G. W. O. Howe. (73) Oct. 7.
The Budapest Telephone Exchange.* (73) Oct. 7.
Design of Armature Coils.* (47) Oct. 8.
Electricity Supply in Willesden.* (73) Oct. 14.
A Recent Wireless Telegraph Patent.* (73) Oct. 14.
The Design of Induction Motors. A. Press. (26) Oct. 14.
Direct Current Transmission at 70 000 Volts.* A. Steens. (26) Oct. 14.
The Present Status of the Edison Storage Battery. A. E. Kennelly and S. E. Whiting. (Abstract of Paper read before the International Elec. Cong.) (73) Oct. 14.
Electrolytic Rectifiers: an Experimental Research. Albert Nodon. (73) Oct. 14.
Steam Turbine Power Plants: Notes on their Equipment and Operation.* J. R. Bibbins. (Paper read before the Amer. St. Ry. Assoc.) (17) Oct. 15; Abstract (13) Oct. 20.
The Problem of the Telephone Relay. John Trowbridge. (From *The Electrical Review*, of New York). (19) Oct. 15.
A Gas Power Plant for the Morecambe Bay Harbor Works.* (14) Oct. 15.
Interior Wiring for Telephones. Charles H. Coar. (27) Oct. 15.
Wireless Telegraphy in Russian Military Field Operations.* Frank C. Perkins. (27) Oct. 15.
The Conduction of Electricity in Mercury Vapour.* A. P. Wills. (From the *Physical Review*.) (73) Oct. 21.
Report on High-Voltage Transmission Lines.* (Committee Rept. to the N. Y. State St. Ry. Assoc.) (73) Oct. 21.
The Lead Voltameter. Anson G. Betts and Edward F. Kern. (Abstract of Paper read before the International Elec. Cong.) (73) Oct. 21.
Rotary Converters and Motor-Generator Sets. Wm. C. L. Eglin. (Abstract of Paper read before the International Elec. Cong.) (73) Oct. 21.
High-Tension Line Construction and Operation. F. A. C. Perrine, M. Am. Inst. E. E. (Abstract of Paper presented before the International Elec. Cong.) (47) Serial beginning Oct. 22.

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* Illustrated.

Electrical—(Continued).

The Water-Wheel Equipment in the Puget Sound Power Company's Plant.* George J. Henry, Jr. (27) Oct. 22.
The Western Electric Company's Exhibit, St. Louis.* (27) Oct. 22.
A French Municipal Electrical Plant.* (27) Oct. 22.
Three-Wire Distribution.* F. Hardie Jeannin. (27) Oct. 22.
On the Theory of the Matthews and the Russell-Léonard Photometers for the Measurement of Mean Spherical and Mean Hemispherical Intensities. Edward P. Hyde. (From *Bulletin* of the Bureau of Standards.) (27) Oct. 22.
Three New Electric Hoists.* (20) Oct. 27.
The Design of Generators for Electric Power Transmission Work.* David B. Rushmore, M. Am. Inst. E. E. (13) Oct. 27.
A Novel Armature Banding Lathe.* (17) Oct. 29.
The Siemens-Schuckert Continuous-Current Wattmeter.* Emile Guarini. (19) Oct. 29.
The Water Power Plants of the Northern California Power Co.* (14) Oct. 29.
Telpherage Plant for Handling Sand.* (27) Oct. 29.
Crane Equipment of the Missouri River Power Station, Kansas City, Mo.* (27) Oct. 29.
Single-Phase Commutator Motors.* Seijiro Sugiyama. (27) Oct. 29.
Resistance Boxes for Use in Precise Alternating Current Measurements. Geo. A. Campbell. (27) Oct. 29.
The Care of Storage Batteries. John Howatt. (64) Nov.
New Powerhouse, West Albany Shops: New York Central & Hudson River Railroad.* (25) Nov.; (64) Nov.
Electrical Progress in Canada. George Johnson. (10) Nov.
Power-House Location.* F. C. Harding. (64) Nov.
Multiple-Voltage Control of Motors.* Norman Gardner Meade. (64) Nov.
L'Énergie Hydro-Électrique, sa Production et ses Applications. Paul Lévy Salvador. (35) Serial beginning Oct.
Telphérage Électrique.* Émile Guarini. (36) Serial beginning Oct. 10.
Usine de la General Electric Company à Schenectady (États-Unis).* (33) Oct. 22.
La Force Motrice à l'Exposition de Saint-Louis: Installation des Machines Motrices.* L. Piaud. (33) Oct. 22.

Marine.

Messrs. Beardmore's New Shipyard and Engine Works.* (11) Oct. 7.
The Auxiliary Power Yacht *Mollihawk II.** (11) Oct. 7.
The Effect of Torpedo Explosions (on the *Sado Maru*).* (11) Oct. 7.
H. M. S. *Black Prince.** (12) Oct. 14.
Protection of Metallic Surfaces (on Vessels). H. Brandon. (Abstract of Paper read before the Inst. of Marine Engrs.) (12) Oct. 21.
Fast Turkish Protected Cruiser *Abdul Hamid.** (19) Oct. 29.
The Proposed 20 000-Ton American Battleships. (10) Nov.
Warships of the Great Powers, a Study of Relative Costs. Archibald S. Hurd. (10) Nov.

Mechanical.

Friction on Lubricated Surfaces.* Fred. A. McKay. (5) v. 17, Pt. 1.
The Practical Use of Extensometers.* K. M. Cameron. (5) v. 17, Pt. 1.
The Economy of Small Gas Engines Using Montreal Illuminating Gas.* Homer M. Jaquays. (5) v. 17, Pt. 1.
The Motion of Gases in Pipes, and the Use of Gauges to Determine the Delivery.* Richard Threlfall. (75) No. 2, 1904.
Mechanical Flight. J. Emery Harriman. (Paper read before the Boston Soc. of Civil Engrs.) (1) Aug.
Simple Steam Turbine Engines.* John Richards. (Paper read before the Technical Soc. of the Pacific Coast.) (1) Sept.
Manufacture and Testing of Portland Cement. C. J. Wheeler. (Paper read before the Technical Soc. of the Pacific Coast.) (1) Sept.
New Process for Utilizing Exhaust Steam. (From *Elektrotechnische Rundschau.*) (70) Oct.
A New System of Vacuum Return Line Exhaust Steam Circulation. James A. Donnelly. (Paper read before The Modern Science Club.) (70) Oct.
The Mt. Holly Brick and Clay Co., Harrisburg, Pa.* (76) Oct.
Air in Relation to Boiler-Feeds.* Jas. Alex. Smith. (Paper read before the Victorian Inst. of Engrs.) (11) Oct. 7.
The Wear of Machinery.* (11) Oct. 7.
Notes on the Manufacture of Coke. John Bell. (Abstract of Paper read before the British Soc. of Min. Students.) (22) Oct. 7.
Coal-Conveying Plant at a Manchester Power Station.* (12) Oct. 7.
Horse-Power Computation for Petrol Motors. C. C. Longridge, M. I. Mech. E. (12) Oct. 7.
Petroleum as Fuel. Boverton Redwood. (From Report of the Royal Commission on Coal Supplies.) (47) Oct. 8.
Variety in Steam Engines. (47) Serial beginning Oct. 8.
Mechanically-Made Cores. George H. Wadsworth. (Abstract of Paper read before the Amer. Foundrymen's Assoc.) (47) Oct. 8.
The New Foundry of the Hilles & Jones Company.* (20) Oct. 18.

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Mechanical—(Continued).

The Calorimetry of Exhaust Gases.* B. Hopkinson. (62) Oct. 13.
Gas, Oil and Steam for Power.* Michael Longridge. (Lecture before the British Assoc. of Mgrs. of Textile Works.) (12) Serial beginning Oct. 14; (47) Serial beginning Oct. 22.
White's Aerial Wire Ropeway.* (11) Oct. 14.
The Hardie-Thompson Water-Tube Boiler.* (12) Oct. 14.
Chimney Action. W. H. Booth. (26) Oct. 14.
Small Curtis Turbines. Richard H. Rice. (Abstract of Paper presented before the Amer. St. Ry. Assoc.) (15) Oct. 14.
Steam Turbine Power Plants: Notes on their Equipment and Operation.* J. R. Bibbins. (Paper read before the Amer. St. Ry. Assoc.) (17) Oct. 15; Abstract (13) Oct. 20.
Steam Turbines.* Richard H. Rice. (Paper read before the Amer. St. Ry. Assoc.) (14) Oct. 15; (17) Oct. 15; Abstract (13) Oct. 20.
High Speed Chain Driving. (14) Oct. 15.
A Pin-Plate and Rivet Diagram.* (14) Oct. 15.
Steam Turbines. (27) Oct. 15.
A Gas Power Plant for the Morecambe Bay Harbor Works.* (14) Oct. 15.
The American Diesel Engine.* E. D. Meier. (Paper read before the Amer. St. Ry. Assoc.) (17) Oct. 15; Abstract (13) Oct. 20.
Sherardizing: A New Process for Protecting Iron and Steel from Corrosion.* (20) Oct. 20.
Flue Gas Testing.* W. H. Whysall. (72) Oct. 20.
A Cable Power Scraper for Earth Excavation.* C. G. Newton. (13) Oct. 20.
High Pressure Steam Pipe Systems: Being a Review and a Comparison of Late Practice on the Continent of Europe and in America.* (72) Oct. 20.
Test of a Reeves Simple Engine.* (72) Oct. 20.
Suction Gas Producers.* (62) Oct. 20.
The Pyrodyen Steam Wagon.* (12) Oct. 21.
Brake Test of a 400-Kilowatt Westinghouse-Parsons Steam Turbine.* (11) Oct. 21.
Design of High-Speed Enclosed Steam Engines.* (12) Serial beginning Oct. 21.
Steam Hammers *vs.* Drop Hammers for Pile Drivers. (Committee Repts. to the Assoc. of Ry. Supts. of Bridges and Buildings.) (40) Oct. 21; (18) Oct. 22; (13) Oct. 27; (15) Oct. 28.
The Future of the Steam Engine. G. F. Charnock. (Lecture before the Bradford Eng. Soc.) (12) Oct. 21; (47) Oct. 22.
Sewer Trenching and Gas Mains. E. G. Smithard. (Abstract of Paper read before the Eastern Counties Gas Managers' Assoc.) (14) Oct. 22.
Air in Relation to Boiler Feed. Jas. Alex. Smith. (Abstract of Paper read before the Victorian Inst. of Engrs.) (14) Oct. 22.
Distributing Power Throughout a Gas Works by Means of Electricity Generated at a Central Point. W. A. Baehr. (Paper read before the Amer. Gas Light Assoc.) (24) Oct. 24.
Concerning Tube Cleaners.* (From *The Locomotive.*) (62) Oct. 27.
Preparation of Alabama Coal for Coke Making. (Abstract of Paper read before the Eng. Assoc. of the South.) (16) Oct. 27.
The Huber Hydraulic Press at the World's Fair.* (20) Oct. 27.
Gasoline-Electric Passenger 'Bus.* (17) Oct. 29.
The Liquid Fuel Investigation of the Navy Department. (14) Oct. 29.
Staybolts. J. J. Ewing. (Paper read before the Richmond Railroad Club.) (18) Oct. 29.
Oxide Purification. E. H. Earnshaw. (Paper read before the Amer. Gas Light Assoc.) (24) Oct. 31.
The Generation of Steam by Waste Heat from Water Gas Sets.* Donald McDonald. (Paper read before the Amer. Gas Light Assoc.) (24) Oct. 31.
Utilization of the Waste Heat from Water Gas Sets for the Generation of Steam.* A. B. Macbeth. (Prepared for the Amer. Gas Light Assoc.) (24) Oct. 31.
The Gas Engines Which Did Not Come (to the Louisiana Purchase Exposition).* Peter Eyerman. (64) Nov.
Experiments with a Chamber for the Production of a Hot Working Fluid to be Used in the Generation of Power.* J. F. Wentworth. (64) Nov.
The Hamilton-Holzwarth Steam Turbine.* (64) Nov.
Compression in Gas Engines.* J. H. Massie. (64) Nov.
Piston Design.* J. F. Nagle. (64) Nov.
Notes on Oil Burners Using Compressed Air. A. P. Hume. (64) Nov.
Strength of Riveted Joints. (64) Nov.
Expansion Joints.* S. T. South. (64) Nov.
The Testing of Steam Engines.* H. Riall Sankey and C. Humphrey Wingfield. (9) Nov.
Modern Types of Plate Planing Machines.* Frank C. Perkins. (41) Nov.
Air Pumps.* W. H. Booth. (64) Nov.
The Adams-Farwell Gasoline Motor.* (41) Nov.; (20) Nov. 3.
Henry & Wright Sensitive Drills.* (20) Nov. 3.
The Putnam 32-Inch High Speed Lathe.* (20) Nov. 3.
Steam Turbines. Richard H. Rice. (Paper read before the Amer. St. Ry. Assoc.) (62) Nov. 3.
The Diesel Engine.* A. W. Oppenheimer. (From *Technics.*) (19) Nov. 5.
Electric Welding.* Emile Guarini. (19) Nov. 5.

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Mechanical—(Continued).

The Steam Turbine in Operation. (14) Nov. 5.
La Régulation des Turbines Hydrauliques.* M. L. Ribourt. (32) July.
Tubes sans Soudure.* M. L. Joubert. (32) July.
Appareils pour l'Épuration de la Vapeur et de son Eau de Condensation.* (37) Serial beginning Oct.
Banc à Étirer pour la Fabrication des Tubes en Fer Soudés à Rapprochement.* (34) Oct.
Note sur les Dispositifs des Joints Entre les Parties Fixes et Mobiles des Élévateurs pour Bateaux. (30) Oct.
Régulateur Automatique d'Alimentation des Chaudières à Vapeur, Système A. Lanne.* R. Michaux. (34) Oct.
Moteurs à Gaz Construits par la Société Française de Constructions Méchaniques. Ch. Dantin. (33) Oct. 8.

Metallurgical.

Ore-Handling Plant at the Clairton Works of the Crucible Steel Company.* Chas. H. Wright. (Paper read before the Civil Engineers' Club of Cleveland.) (1) Aug.
Surface Flow and the Hard and Soft State in Metals. (11) Oct. 7.
Ore Concentration Plant at the St. Louis Exhibition.* (11) Oct. 7.
The Rolling of Sections in Iron and Steel.* Adolph S. While. (20) Oct. 13.
Electric Furnace for Manufacturing Steel.* (47) Oct. 15.
The Manufacture of Steel Castings. W. A. Herron. (Abstract of Paper read before the Ry. Club of Pittsburg.) (47) Oct. 15.
Gayley Method for Extracting Moisture from Air for Furnace or Converter.* (62) Oct. 20.
The Chemistry and Metallurgy of Copper. Charles S. Palmer. (16) Serial beginning Oct. 20.
The Huntington-Heberlein Process.* Donald Clarke. (16) Oct. 20.
Ore Treatment at the Ivanhoe Mine, Kalgoorlie. H. J. Brooke. (16) Oct. 20.
A California Chlorination Plant.* (16) Oct. 20.
Modern Methods of Steel Casting.* Joseph Horner, A. M. I. Mech. E. (19) Serial beginning Oct. 22.
Chlorine in Metallurgy. James Swinburne. (Abstract of Paper read before the International Elec. Cong.) (47) Oct. 22.
Chlorination in Colorado.* William E. Greenawalt. (16) Oct. 27.
A New Ore-Testing Plant.* (16) Oct. 27.
The Bradford-Carmichael Process.* Donald Clark. (16) Nov. 3.
Evaporation of Solutions. (16) Nov. 3.
The Molecular Changes from Pig Iron to Steel. A. D. Elbers. (62) Nov. 3.
Propriétés, Classification et Utilisation des Aciers Spéciaux Ternaires.* M. L. Guillet. (32) July.

Military.

Transportable Wireless Telegraph Station for War Purposes.* (46) Oct. 15.
British Hydraulic Machinery: Its Use in the Manufacture of Munitions of War.* A. F. Petch. (10) Nov.

Mining.

The Industries of the Consolidated Lake Superior Company.* E. G. M. Cape. (5) v. 17, Pt. 1.
On the Methods of Working Coal in Saxony and Austria.* (22) Oct. 7.
Coal Winding Machinery.* Horace D. B. How. (From the *Transactions* of the South Wales Inst. of Engrs.) (57) Oct. 7.
Power Transmission by Compressed Air in Mines. R. W. Chapman. (Paper read before the Australian Inst. of Min. Engrs.) (47) Oct. 8.
Stamp Tappets.* M. P. Boss. (16) Oct. 13.
A Great Engineering Problem.* W. A. Lawson. (16) Oct. 13.
Electrically Driven Air Compressors (for Mining Purposes). (From *Bulletin 101*, pub. by Harron, Rickard & McCone.) (16) Oct. 13.
A Modern Method of Coal Washing. C. A. Meissner. (Abstract from the *Journal* of the Mining Soc. of Nova Scotia.) (16) Oct. 13.
The Mines and Mineral Industries of Italy.* Enrico Bignami. (9) Nov.
The Tamarack Hoisting Engine.* (64) Nov.
Coal Working in Saxony.* (16) Nov. 3.
A Concrete-Lined Shaft for the U. S. Coal & Coke Co.* Francis Donaldson. (13) Nov. 3.
Visite de la Société des Ingénieurs Civils de France aux Mines d'Anzin.* Maurice Boutté. (32) Aug.
Notice sur la Compagnie des Mines de Houille de Marles (Pas-de-Calais).* M. E. Suisse. (32) Aug.
Les Nouveaux Evite-Molettes à l'Exposition d'Arras.* H. Schmerber. (33) Serial beginning Oct. 22.

Miscellaneous.

Engineering at Guy's Hospital. (12) Serial beginning Oct. 14.

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Municipal.

Losses in Underground Municipal Structures. A. Prescott Folwell. (Paper read before the Amer. Soc. of Mun. Improvements.) (14) Oct. 15; (60) Nov.
A Municipal Laboratory for Testing Asphalt and Cement. W. H. Broadhurst. (Paper read before the Amer. Soc. of Mun. Improvements.) (14) Oct. 15.
Street Lighting. A. S. Hatch. (Paper read before the International Assoc. of Mun. Electricians.) (24) Oct. 17.
A New English Telephonic Fire Alarm.* Frank C. Perkins. (46) Oct. 22.
The Time Required to Raise Steam for Fire Pumps from Cold Boilers and the Cost of Banked Fires.* (13) Nov. 3.
Some Road Building in the Philippines.* H. L. Stevens. (14) Nov. 5.
La Construction des Villes.* L. Cloquet. (31) Pt. 3, 1904.

Railroad.

The Kilanea Plantation Railway.* G. R. Ewart. (5) v. 13, Pt. 2.
Transition Curves. E. S. M. Lovelace. (5) v. 13, Pt. 2.
Railway Fencing. R. W. Leonard. (5) v. 17, Pt. 1.
Some Theories upon Railroad Location.* J. G. G. Kerry. (5) v. 17, Pt. 1.
Compound Locomotives in France.* Edouard Sauvage, M. I. Mech. E. (75) No. 2, 1904.
Vertical Railway Curves.* H. I. Randall. (Paper read before the Technical Soc. of the Pacific Coast.) (1) Sept.
The Relative Merits of Large Grates and Heating Surfaces and Their Proportions.* (Abstract of Com. Rept. to the Amer. Ry. Master Mechanics' Assoc.) (61) Sept. 20.
Negative Work of Back Pressure and Compression.* Ira C. Hubbell. (61) Sept. 20.
Square Engine Houses: Terminal Railroad Association of St. Louis.* (39) Oct.; (18) Oct. 15.
Radial Staybolts for Oil Burning Engines.* I. C. Hicks. (39) Oct.
The Schenectady Super-Heater for Locomotive Practice.* (39) Oct.
Ten Wheel Passenger Locomotive with Wide Firebox Boiler: C. B. & Q. Lines West.* (39) Oct.
Painting Steel Cars, and Removing Cracked Varnish. (Reports of Committees and Papers read before the Master Car and Locomotive Painters' Assoc. of the U. S. and Canada.) (39) Oct.
What Is the Best Material and Treatment for Locomotive Front Ends? (Papers and Report of Committee read before the Master Car and Locomotive Painters' Assoc. of the U. S. and Canada.) (39) Oct.
Flange Wear and Side Bearing Trucks.* Gustav Lindenthal. (65) Oct.
Motor-Bogie for Electric Trains.* (11) Oct. 7.
Consolidation Engine. Tested at the St. Louis Exhibition.* H. W. Hanbury. (11) Oct. 7.
Motor Cars for Light Passenger Traffic on Steam Railways.* (13) Oct. 13.
Circuits for Manual Block Systems.* (Committee Rept. to the Ry. Signal Assoc.) (40) Oct. 14.
Track Depression of the Chicago & Northwestern Railway at Milwaukee.* (40) Oct. 14.
Notes on Equipment of the Wilkesbarre and Hazleton Railway.* Lewis B. Stillwell. (Abstract of Paper read before the International Elec. Cong.) (73) Oct. 14.
Express Passenger Engine, London and North-Western Railway.* (12) Oct. 14.
Electric Goods Locomotive.* (11) Oct. 14.
Western Union Railway of Australia.* (12) Oct. 14.
The Illmo Yards of the St. Louis Southwestern.* (15) Oct. 14.
The Stansstad-Engelberg Electric Railway.* (15) Oct. 14.
The St. Louis, Memphis & Southeastern Railroad.* (15) Oct. 14.
Pacific Type Locomotive, Southern Pacific.* (18) Oct. 15.
Economy in the Use of Steel Concrete (Through a Railroad Embankment). John B. Warren. (14) Oct. 15.
Standard Specifications for Mechanical Interlocking Material and Construction Work. (Committee Rept. to the Ry. Signal Assoc.) (18) Oct. 15; Abstract (13) Oct. 20.
The St. Louis, Kansas City & Colorado Railroad. Now the St. Louis Division of the Rock Island System.* J. L. Campbell, M. Am. Soc. C. E. (13) Serial beginning Oct. 20; (15) Oct. 21.
The Cole Balanced Compound Locomotive at the St. Louis Exhibition. H. W. Hanbury, A. M. Inst. C. E. (11) Oct. 21.
Freight and Roundhouse Doors and Fittings. (Committee Rept. to the Assoc. of Ry. Supts. of Bridges and Buildings.) (40) Oct. 21; (18) Oct. 22; (13) Oct. 27.
Railway Sleepers. (12) Oct. 21.
Cost of Concrete Structures. H. M. Jones. (Abstract of Paper presented to the Eng. Assoc. of the South.) (15) Oct. 21.
New Compound Locomotives in Bavaria.* Charles R. King. (15) Oct. 21.
Cost of Ditching Cuts and Widening Embankments.* (Abstract of Com. Rept. of the Roadmasters' and M. of W. Assoc.) (15) Oct. 21; (13) Oct. 27.
Three-Phase Alternating Current for Electric Railways. A. Zelewsky. (Abstract of Paper read before the International Elec. Cong.) (15) Oct. 21.
Preliminary Railroad Surveys by the Stadia Method.* J. A. MacDonald. (15) Oct. 21.
Turning Steel-Tired Car Wheels at the Readville Shops.* (40) Oct. 21.

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Railroad—(Continued).

Recent Improvements in Block Signals. C. H. Platt. (Abstract of Report No. 1 for Discussion before the International Ry. Cong.) (40) Oct. 21.
Delaware, Lackawanna & Western Locomotive and Car Repair Shops at Kingsland, N. J.* (40) Oct. 21.
Construction Difficulties on the Guayaquil & Quito Railway.* (14) Oct. 22.
Circuits for Manual Block Systems.* (Committee Rept. to the Railway Signal Assoc.) (18) Oct. 22; Abstract (13) Oct. 20.
Keyser Valley Shops, D. L. & W. R. R.* (18) Oct. 22; (15) Oct. 28.
Protecting Low Structures over Tracks from Locomotive Gases and Blasts. (Committee Rept. to the Assoc. of Ry. Supts. of Bridges and Buildings.) (13) Oct. 27; (15) Oct. 28; (18) Oct. 29.
Automatic Block Signals. M. Margot. (From *Bulletin* of the International Ry. Cong.) (40) Oct. 28.
Specifications for Air-Brake Hose.* M. H. Wickhorst. (Abstract of Paper read before the Amer. Soc. for Testing Materials.) (15) Oct. 28.
The Haberkorn Valve Mechanism for Locomotives.* (15) Oct. 28.
Passenger Locomotive with Vanderbilt Tender.* (46) Oct. 29.
The Use of Soda Ash in Locomotive Tanks. S. K. Dickerson. (Paper presented before the Central Railway Club.) (18) Oct. 29.
Cost of Concrete Construction on the Nashville, Chattanooga & St. Louis Ry. H. M. Jones. (Abstract of Paper read before the Engineering Assoc. of the South.) (14) Oct. 29.
The Works of the Locomotive & Machine Co. of Montreal.* (14) Oct. 29.
New Shops of the Pere Marquette R. R. at Grand Rapids, Mich.* (18) Serial beginning Oct. 29.
Oil Burning Consolidation Locomotive, Southern Pacific Co.* (18) Oct. 29.
Electric Locomotive Hoist, Atchison, Topeka and Santa Fe Railway.* (25) Nov.
Variable Speed Motors in Railway Machine Shops.* J. C. Steen. (25) Nov.
Painting of Steel Cars. (Committee Rept. to the Master Car and Locomotive Painters' Assoc.) (25) Nov.
Two Remarkably Efficient Wood-Working Tools, Grand Trunk Railway.* (25) Nov.
Roundhouse Cranes and Smoke Jacks.* (25) Nov.
The Building of a Chinese Railway.* Justin Burns. (9) Nov.
New Powerhouse, West Albany Shops: New York Central & Hudson River Railroad.* (25) Nov.; (64) Nov.
Powerful Prairie Type Passenger Locomotives: Lake Shore & Michigan Southern Railway.* (25) Nov.
Ball Bearing Center Plates and Side Bearings.* (25) Nov.
The Summers Gravity Dump Car.* (25) Nov.
Highest Development of Air Brakes, the High Speed Brake. (25) Nov.
The Washington Terminal Improvement: A General Description of the Union Station and the Elimination of Grade Crossings in Washington.* W. F. Strouse. (13) Nov. 3.
Great Northern Terminal Improvements at Seattle.* (15) Nov. 4.
The Young Valve and Gear for Locomotives.* (15) Nov. 4.
Train Resistance.* Chester A. Crandell. (15) Nov. 4.
The Railroad Cross-Tie Problem.* S. Whinery. (15) Serial beginning Nov. 4.
An Electric Freight Locomotive for the Philadelphia & Reading Railroad. (17) Nov. 5.
An All-Steel Car Framing.* (17) Nov. 5.
Energy Required in Watt-Hours when Braking with Air. Frank B. Rae. (17) Nov. 5.
The Use of Independent Motor Cars on Railways.* Philip Dawson. (17) Nov. 5.
Description d'une Cabine d'Enclenchements à Trajecteurs Pneumatiques à Basse Pression mise à l'Essai par la Compagnie du Nord, dans la Gare d'Ermont.* M. E. Despons. (38) Oct.
Rouleau Compresseur à Vapeur. (34) Oct.
Le Tunnel du Simplon. L. Faullain de Banville. (36) Oct. 10.
Tramway Électrique avec Mécanisme d'Adhérence à Roues Horizontales et Rail Central en Service à La Bourboule (Puy-de-Dôme).* A. Dumas. (33) Oct. 22.

Railroad, Street.

Notes on Solid Rail Joints. Percy S. Sheardown, A. M. I. E. E. (77) Sept.
Methods of Dealing with Mixed Systems of Traction. A. L. C. Fell. (Paper read before the Mun. Tramways Assoc. of Great Britain.) (73) Oct. 7; (47) Oct. 15.
New Swiss Electric Railway.* (From *Schweizerische Elektrotechnische Zeitschrift.*) (73) Oct. 7.
South Lancashire Tramways.* (26) Serial beginning Oct. 7.
Overhead Equipment: Notes on its Construction and Maintenance. J. M. McElroy. (Paper read before the Mun. Tramways Assoc.) (73) Oct. 7; Abstract (26) Oct. 7; (47) Oct. 15.
Cars: Their Equipment and Maintenance. John Aldworth. (Abstract of Paper read before the Mun. Tramways Assoc. of Great Britain.) (47) Oct. 8.
The Electric Railways of St. Louis.* (17) Oct. 8.
The Evolution of the (Street Railway) Industry.* (17) Oct. 8.
The New York Rapid Transit Subway.* (17) Oct. 8.
Power Plant of the Intramural Railway at the St. Louis Exhibition.* (13) Oct. 13.

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Railroad, Street—(Continued).

The Harlem River Tunnel Section of the New York Rapid Transit Railway.* (13) Oct. 18.
Worm Reduction in Electric Tramway Equipments.* O. F. Domon, M. I. E. E. (26) Oct. 14.
Sprague-General Electric Automatic Relay Train Control for Boston Elevated Railway.* G. H. Hill. (27) Oct. 15.
Steam-Turbine Power Plants.* (14) Oct. 15.
The Mono-Rail. F. B. Behr. (Abstract of Paper read before the International Cong. of Elec. Engrs.) (17) Oct. 15.
The Ideal Shop. W. D. Wright. (Paper read before the Amer. Ry. Mech. and Elec. Assoc.) (17) Oct. 15; (13) Oct. 20.
The Mandalay Electric Tramways.* (17) Oct. 15.
Wheel Matters.* J. Millar. (Paper read before the Amer. Ry. Mech. and Elec. Assoc.) (17) Oct. 15; Abstract (13) Oct. 20.
Solid Forged Steel Trucks.* (18) Oct. 15; (62) Nov. 3.
Exhibit of the Westinghouse Traction Brake Company at the World's Fair.* (18) Oct. 15.
Freight and Express Business of the Interurban Railway & Terminal Co. (72) Oct. 20.
The Determination of Schedules by Speed Templates.* Howard S. Knowlton. (72) Oct. 20.
Test Car of the Great Tramway Co. of Berlin.* (72) Oct. 20.
Some Single-Phase Railway Problems. Paul M. Lincoln. (Paper read before the International Elec. Cong.) (73) Oct. 21.
The Sot's Road Generating Station of the Underground Electric Railways Co. of London.* (73) Oct. 21.
Rolling Stock on the Leek and Manifold Light Railway.* (12) Oct. 21.
Power House of the New York Rapid Transit Subway.* (73) Serial beginning Oct. 21.
Single-Phase Electric Traction. Bion J. Arnold. (Abstract of Paper read before the International Elec. Cong.) (40) Oct. 21.
Rapid Transit Railway Construction in Brooklyn.* (14) Serial beginning Oct. 22.
The Territory, Construction, Power Transmission, System and Rolling Stock of the Oregon Water Power & Railway Company.* (17) Serial beginning Oct. 22.
The Work of the Electric Railway Test Commission.* Henry H. Norris. (17) Oct. 22.
A New Design of Snow Plow, Hartford Street Railway Company.* (17) Oct. 22.
The Effect of Changing the Gear Ratio on a Series Railway Motor Equipment.* J. C. Huffman. (17) Oct. 29.
Bitulithic Pavement in Street Railway Tracks.* (14) Oct. 29.
Opening of the New York Rapid Transit Subway.* (46) Oct. 29.
A New Design of Steel Motor Truck for Heavy Electric Traction Service.* (17) Oct. 29.
New Cars for Elevated Service, Boston Elevated Railroad.* (25) Nov.
Steam Motor Cars used on European Interurban Railways.* (17) Nov. 5.
A New Type of American Double-Deck Car.* (17) Nov. 5.
Changes in German Electric Railway Practice During the Past Year. (17) Nov. 5.
The Westinghouse Single-Phase, A. C. Railway System for Interurban Service.* (17) Nov. 5.
High-Tension Trolley Insulators.* (17) Nov. 5.
La Traction par Courant Alternatif Simple, Système Winter-Eichberg.* (33) Oct. 8.

Sanitary.

The Reclamation of a Mountain Swamp.* Marsden Manson. (Paper read before the Technical Soc. of the Pacific Coast.) (1) Sept.
The Heating of Suites of Rooms by Warm Water.* Alfred Luci. (From *Wärme und Heizung*.) (70) Oct.
Some Odd Types of Ventilating Fans.* (70) Oct.
Plumbing a Police Station.* (70) Oct.
The Royal Commission on Sewage Disposal. (12) Serial beginning Oct. 7.
A Concrete-Steel Sewer at Harrisburg, Pa.* (14) Oct. 15.
The Sewage Testing Station at Columbus, O.* John H. Gregory, Assoc. M. Am. Soc. C. E. (13) Oct. 20.
Scientific Disposition of Sewage. Charles F. Holder. (46) Oct. 22.
The Use of Sulphur for Sewer Pipe Joints in Wet Trenches. E. S. Rankin. (60) Nov.
The Pollution of the Passaic River and Its Prevention. Ernest Adam. (Paper read before the Amer. Soc. of Mun. Improvements.) (60) Nov.
Sewage Disposal for Small Cities and Towns. T. Chalkley Hatton. (Paper read before the Amer. Soc. of Mun. Improvements.) (60) Nov.
Sewage Disposal. J. N. McClintock. (Paper read before the Amer. Soc. of Mun. Improvements.) (60) Nov.
Some Experiments with Ventilating Fans.* D. W. Taylor. (13) Nov. 3.
The Disposal of Sewage Sludge at Saltley, England. (14) Nov. 5.

Structural.

The Measurement of Direct Strains in Tensile and Compressive Test-Pieces.* John Morrow. (75) No. 2, 1904.
Concrete-Steel.* M. C. Couchot. (Paper read before the Technical Soc. of the Pacific Coast.) (1) Sept.

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Structural—(Continued).

A New Method of Building Construction.* S. Giletti. (Paper read before the Technical Soc. of the Pacific Coast.) (1) Sept.
Manufacture and Testing of Portland Cement. C. J. Wheeler. (Paper read before the Technical Soc. of the Pacific Coast.) (1) Sept.
Paving Brick for Residences and Public Buildings.* (76) Oct.
The Classification of Iron and Steel. Albert Sauveur. (Paper read before the Amer. Soc. for Testing Materials.) (47) Oct. 8.
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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

PROBABLE WIND PRESSURE INVOLVED IN THE WRECK OF THE HIGH BRIDGE OVER THE MISSISSIPPI RIVER, ON SMITH AVENUE, ST. PAUL, MINN., AUGUST 20TH, 1904.

By C. A. P. Turner, M. Am. Soc. C. E.

To be Presented December 7th, 1904.

In view of the fact that the wreck of a well-braced iron or steel structure by wind is exceedingly rare, if, indeed, there is any previous record of such, the destruction of part of the so-called High Bridge over the Mississippi River at Smith Avenue, St. Paul, would seem to be of special interest to the professional bridge engineer.

This structure, Plate XLII, was designed supposedly to meet, with a reasonable factor of safety, the maximum wind loads required by a standard specification; and the utter destruction of a portion of it by wind pressure alone, in view of this supposed margin of safety, would lead to the presumption that the standard requirements do not produce a safe structure, unless it can be shown by reasonable computation that there was some weak joint or detail in the frame which

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SMITH AVENUE VIADUCT, ST. PAUL, MINN., OVER MISSISSIPPI RIVER, VIEW FROM THE UP-STREAM SIDE, BEFORE THE STORM OF AUGUST 20TH, 1904.

would insure its destruction under the action of forces not materially greater than those which, nominally, it was designed to withstand.

The structure was built in 1887, according to general plans prepared by the City Engineering Department of St. Paul. Detailed drawings were made by the Contractor, C. L. Strobel, M. Am. Soc. C. E., and the work was erected by Horace E. Horton, M. Am. Soc. C. E., of Chicago, Ill.

The bridge is a deck structure of wrought iron, 2 770 ft. long, and runs northwest and southeast. The northwest portion of the bridge is of the viaduct type, with riveted spans of 80 ft. and plate-girder tower spans of 40 ft. Four-leg towers alternate with two-leg bents. The portion of the viaduct over the river consists of four 250-ft. pin-connected deck spans of the subdivided Warren type, 30 ft. deep and 22 ft. from center to center of trusses. The floor beams are at 12 ft. 6 in. centers. The tower supporting the shore end of the southeast 250-ft. span has a base of 55 ft. transversely, and of 50 ft. longitudinally, and a height of 129 ft. from the top of the pier to the bottom chord of the truss. As these trusses were 30 ft. deep, the roadway at this point was 160 ft. above the pier and about 180 ft. above the water From this tower toward the bluff there was one 170-ft. pin span and two 60-ft. plate-girder spans.

These girder spans, the 170-ft. pin span, the supporting tower, and the 250-ft. pin span were overthrown, as shown in Plates XLIII, XLIV and XLV.

The bridge carries a 25-ft. roadway and two 8-ft. walks. The flooring for the roadway consists of a sub-floor of $3\frac{3}{4}$-in. fir plank and a wearing floor $1\frac{3}{4}$ in. thick. The plank for the walks is $2\frac{3}{4}$-in. pine. The stringers are of steel, the roadway of nine lines of 12-in. built stringers; the flanges are each two L's, 2 by 2 by $\frac{5}{16}$-in., with $\frac{5}{16}$-in. webs. The stringers for the walks are 6-in. I's.

The trusses were designed for a live load of 80 lb. for the roadway and for the walks of the 250-ft. spans, 90 lb. for the 170-ft. span and 100 lb. per sq. ft. for all shorter spans.

The lateral bracing was designed for a pressure of 450 lb. per lin. ft. of bridge, two-thirds of which was assumed to act on the loaded (upper) chord. The towers and bents were assumed to have a wind pressure of 150 lb. per lin. ft. acting against them.

In addition to the top and bottom lateral systems, a fairly efficient system of sway rods was provided in the 250-ft. span, and all the details of the lateral and sway bracing seem to have been well worked out for the type of bracing used.

Referring to Figs. 1 and 2, Plate XLIII, it will be noted that the 250-ft. span is lying on its side, except at the end torn from its support on the two-leg bent (still standing), and that this end has been given a quarter twist in addition and has fallen or has been blown a considerable distance from the tower.

Referring to Figs. 1 and 2, Plate XLV, the plate-girder spans seem to have been pulled down the bank, and are but little out of the line of the viaduct.

The tower frame which was overthrown was badly twisted, and the position in which the columns fell, together with the manner in which the bolts were bent and broken, would seem to indicate that the end of the 250-ft. span resting on the two-leg bent was first pushed off its support, and that the wind, acting on the loose span with its 10 000 or 11 000 sq. ft. of exposed area (the planking was well fastened) and an extreme leverage of 250 ft., twisted from its base the tower bent supporting the other end, and the falling mass, in its descent, pulled the girder spans down the hill.

If the collapse occurred as outlined, a few figures on the twisting moment on the top of the tower may be in order. Supposing the floor to be at such an angle to the wind that the effective pressure is, say, 10 lb. per sq. ft., then the twisting moment $= 10 \times 10\,000 \times 125$ ft. $=$ 12 500 000 ft-lb., an amount far in excess of the ultimate resistance of the tower.

The next point which would seem to invite attention is the detail of the connection of the end of the wrecked 250-ft. span to the two-leg bent, and the strength, or resistance of this connection to uplift and to lateral sliding of the shoe.

Referring to Figs. 1 and 2, Plate XLIII, it will be noted that there is a two-leg bent, similar to the one that supported the end of the wrecked span, nearer the other shore, and, as this seat was easily accessible by a trap in the floor, a suspended platform and an iron ladder leading down to the shoe, it was examined first. The end of the 250-ft. span corresponding to that which was wrecked was found to rest on a nest of eight rollers each about $2\frac{3}{4}$ in. in diameter with the usual spacing bars on the sides.

The sole resistance to the lateral motion of the rollers was a bar riveted to the cap on each side and a recess in the shoe above, about $\frac{1}{4}$

FIG. 1.—SMITH AVE. VIADUCT. VIEW LOOKING NORTHWEST. WRECKED BY STORM OF AUGUST 20TH, 1904.

FIG. 2.—SMITH AVE. VIADUCT. VIEW LOOKING NORTHWEST

or $\frac{5}{8}$ in. in depth, as nearly as could be readily determined. Provision was made for a $1\frac{1}{4}$-in. guard bolt on each side of the shoe, a hole was provided in the column cap and a long slot in the side of the shoe. No bolts, however, were in place. The photograph, Fig. 2, Plate XLVI, taken from the platform vertically above the shoe, shows clearly the hole where this bolt should have been, on the outside of the shoe, on the up-stream side of the bridge; and Fig. 1, Plate XLVI, shows the absence of the bolt on the down-stream side. The inner sides of the shoes could not be photographed conveniently, but the bolts were missing there also.

Fig. 2, Plate XLVII, is a photograph of the column cap from which the 250-ft. span slid off on the leeward side of the bridge, and Fig. 1, Plate XLVII, is a view of the cap on the windward side. Each of these views was taken looking diagonally downward from the end of the floor still standing. It may be noted that but one roller has been left on the windward cap while seven of the eight remain on the leeward cap. Careful examination of these photographs will show that this span was anchored down somewhat better than the one referred to above, and instead of having no bolts at all, there was one on the outside of the windward shoe which is splintered and broken in place. The appearance of the other three holes is positive evidence that there were no bolts in any of them.

There is, then, the resistance of this end of the span, reduced to the dead weight, and the value of this bolt. If the wind tended to raise the windward truss, as it is pin-connected with the eye-bar bottom chord and diagonals, the truss would furnish little resistance to upward forces, and the bolt at the end, being a cantilever from $3\frac{1}{2}$ to 4 in. from the center of the shoe plate to the center of the bearing in the cap, would not develop its shear value, but only its bending value, the insignificant amount of 3 000 or 4 000 lb. or less.

An uplift on the windward side would be accompanied by a reversal of stress in the bottom chord, the probable buckling of the chord, and, with the slight resistance of the bolt, the shoe would be pulled from the cap and the rollers displaced, as appears in Fig. 1, Plate XLVII.

A rough approximate estimate of the weight of the span and floor would be in the neighborhood of 2 200 lb. per lin. ft., giving a reaction of, roughly, 140 000 lb. at each support. As the storm was a severe one, it will be assumed, for purposes of computation, that the very severe wind pressure of 30 lb. per sq. ft. was acting at an upward angle to the floor of 30°, and the pressures will be calculated in accord with Unwin's table.

Let $a =$ Angle of surface with direction of wind;

$F =$ Force of wind, in pounds per square foot (assumed at 30 lb. per sq. ft.);

$A =$ Pressure normal to surface $= F \sin. a^{1.84 \cos. a - 1}$;

$C =$ Pressure parallel to direction of wind $= F \sin. a^{1.84 \cos. a}$.

For $a = 30°$, $A = 0.66$ and $C = 0.33$.

The direct uplift at each shoe $= 20.5 \times 125 \times 0.66 \times 30$ lb. $=$ 50 100 lb. from the wind on the floor.

The overturning force, C, at each end of the bridge $= 0.33 \times 30$ lb. $\times$ $41 \times 125 = 51\,200$ lb. from the wind on the floor.

The uplift from C on the windward shoe $= \dfrac{51\,200 \times 30}{22} = 70\,000$ lb.

The direct pressure on the side of the truss top chord, approximately, $= 30 \times 7 = 210$ lb. per lin. ft., and $\dfrac{210 \times 125 \times 30}{22} = 36\,000$ lb., the uplift from the same.

As the wind has been assumed to be blowing upward, this component on the vertical area would give an additional uplift of some 6 000 lb.

Now, the sum of these computed uplifts is 162 100 lb., or about 15% greater than the reaction due to weight.

Evidently, if the windward shoe is raised, there being no bolts to hold down the leeward shoe, it would turn sufficiently for the recess in the shoe to clear the corner of the rolls and then slide off the cap.

Allowing some slight resistance for the expansion connection of the stringers to the beam, it would seem safe to conclude that the wind pressure assumed is 10% greater than would have been necessary to cause the wreck.

Rough computations on the laterals, taking into consideration the sway rods and the action of the four planes of bracing, would indicate that they were not strained much beyond 23 000 to 25 000 lb. under the assumed forces.

Bearing in mind the fact that the floor is on a steep up grade, it may well be that the angle of action of the wind on the floor was greater than has been assumed, and, if so, the necessary pressure to

FIG. 1.—SMITH AVE. VIADUCT. VIEW LOOKING SOUTHWEST. SHOWING PART OF SHORE TOWER, AND ALSO RAILROAD TRESTLE CUT THROUGH BY THE FALLING TOWER.

FIG. 2.—SMITH AVE. VIADUCT. LOOKING UP STREAM. SHOWING SECTION OF VIADUCT THROWN DOWN BY STORM.

cause the wreck might be considerably less than the 27 lb. per sq. ft. calculated. Again, the probability is that the assumption of a uniform pressure is materially in error. Judging somewhat by the contour of the bluff and the path of the storm, it would seem likely that the maximum pressure was in the vicinity of the northwest end of the wrecked 250-ft. span, and that the adjoining span was saved by its rigid connection to the two-leg rocker bent. If the pressure were greater at the end, it is evident that the average pressure necessary to cause the wreck would be materially less.

Such moderate pressures as have been figured on, when their cumulated effect is concentrated upon a weak detail, may evidently produce results that cause astonishment, and the rash assumption, by those whose training should lend better judgment, that the pressures involved are "exceedingly great."

Evidently, whether dealing with bridges or roofs, stiff riveted construction, with bottom chords and diagonals capable of taking reverse stresses is to be preferred, and, in view of the fact that, by the exercise of reasonable skill in design, they can be fabricated for a sufficiently smaller cost than the pin type to offset the additional expense of riveting in the field, they should be preferred for all moderate spans, such as 250 ft. or less, unless the work is exceptionally heavy.

In the provision for temperature stress, the expensive and frequently weak details often worked out to avoid a harmless little amount of temperature strain, in an effort to eliminate it entirely, is indeed surprising; perfect double-action joggle connections are too often introduced at the shortest possible intervals, and dignified by the name of expansion joints.

In the present instance, for example, an ordinary sliding plate fitted with a compression grease cup would probably move as easily as the badly rusted rolls on a rusty base and cap; while the guard bolts would be brought into actual shear and tension under forces tending to displace the shoe instead of inbending as with the detail adopted.

From a careful examination of the 2¼-in. anchor bolts of the windward column of the fixed bent under the 250-ft. span, it would seem that they were without nuts, though this fact appears to have had no material influence on the wreck.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE RECLAMATION OF RIVER DELTAS AND SALT MARSHES.

By J. Francis Le Baron, M. Am. Soc. C. E.

To be Presented December 21st, 1904.

In the following paper an attempt is made to show the practicability and great desirability of reclaiming the immensely rich swamp lands situated at the mouths of many rivers, notably the Mississippi, and the salt marsh lands lying along the whole seaboard, lands than which no richer exist on the continent, and which are pre-eminently adapted to most successful cultivation of, not only rice, but sugar and all classes of garden truck, these reclaimed swamp lands being of much greater agricultural value and capability than the irrigated prairies, which are so popular at present with rice growers in Louisiana and Texas.

The lands of the Mississippi Delta, being probably the largest single body of fresh and salt marshes in the United States, have been selected for examination and study, as being without doubt the most difficult of treatment, on account of the low range of the tides and the elevation of the river above the land to be reclaimed.

In carrying out the plan of explaining the treatment of the most

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PLATE XLV.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1904.
TURNER ON
WIND PRESSURE ON BRIDGES.

FIG. 1.—SMITH AVE. VIADUCT. VIEW LOOKING NORTHEAST, SHOWING PARTS OF SHORE TOWER, 170-FT. SHORE SPANS AND TWO 60-FT. PLATE GIRDERS.

FIG. 2.—SMITH AVE. VIADUCT. VIEW FROM END OF VIADUCT STILL STANDING, SHOWING PART OF WRECK AND ABUTMENT OF WRECKED END.

difficult cases, from which it is easy to change the *modus operandi* to suit more advantageous conditions, the writer has based his computations on rice culture, as requiring more irrigation water, and consequently more pumping, to irrigate or de-water. The plans can be readily modified to suit other conditions and localities.

The writer has lately been engaged to make examinations and to report on the reclamation of 500 000 acres of the Mississippi Delta lands.

Location and Character of Lands of the Mississippi Delta.

The lands selected for discussion include the salt and fresh marshes and swamp lands in the Parishes of Plaquemines, St. Bernard, Jefferson, La Fourche, Terrebonne, Orleans, St. Charles, St. Mary, Iberia, St. John the Baptist, St. James, Assumption and Ascension. The conclusions and methods are also applicable to large portions of Vermilion and Cameron, and the lower parts of the Louisiana and Florida parishes, as well as all deltas and salt marshes.

Most of these lands are open fresh marsh, merging gradually into salt marsh at the southern end, and covered only with grass. They are entirely free from trees or bushes, except for a narrow margin along the bayous and a few scattering "chênières," or oak islands, and ridges in the marshes, which in Florida would be called "hammocks."

These lands are from 6 in. to 10 ft. above Mean Gulf Level, the great majority being about 2 ft.; and their drainage must be effected by pumping, as is the general custom in this region, where too low to drain by gravity.

The soil is composed of the rich alluvium brought down and deposited by the Mississippi River during past ages, and is inexhaustible in fertility. Probably no richer agricultural soil exists on the Continent of North America.

The writer caused numerous borings to be made, and tested the soil, personally, in several places. The borings, made for the New Orleans and Gulf Ship Canal and Locks, show it to consist of black clay, sand and silt for a depth of more than 80 ft., or as far down as the borings extended, intermixed in varying proportions, the silt being composed largely of vegetable matter. The State Commissioner of Agriculture says of these parishes: "The soil is exceedingly rich and productive."

Methods of Rice Cultivation.

In the cultivation of rice in the United States, two methods are now followed, the older being that in vogue in the lowlands of the Carolinas and Georgia. The modern method is radically different, and was first essayed in the new rice fields on the elevated prairies of western Louisiana and eastern Texas.

In the first case, large quantities of water are used in irrigating the rice, and are considered absolutely necessary, the fields being low, swampy, and having to be embanked to keep them from being overflowed. In the later or modern method, comparatively small quantities of water are used. It is all pumped or obtained from artesian wells, and is used to irrigate the dry prairies. These fields are embanked to keep the water in. In the first case, the irrigation water is put on the fields by gravity, and, in most cases, has to be pumped off. In the latter, the water is generally pumped on, and is drawn off by gravity.

By the old plan, the watering extends over 95 days, whereas the modern practice proves that 68 days is sufficient, if the water is put on at the right time.

Methods of Reclamation.

The methods pursued in reclaiming land for rice, cane and vegetables are essentially the same. The important thing is to control the water supply and the drainage, protect the land from the overflow of salt water and crevasse water by ample protection levees, and make the lands long and narrow so that they can be worked by machinery and yet be well drained.

Lands close to the Mississippi River can be reclaimed naturally at less cost than those at a greater distance, as the irrigating water can be taken over the levee in siphons and put on the land with little trouble by short ditches. For lands farther away, wooden flumes must be built, in some cases, to cross intervening bayous, and, in some instances, pipes must be laid. As long as the supply is taken from the Mississippi River, either by siphons or by pumps, there will be no lack of water for these plantations, but the quantity to be pumped will be influenced largely by the rainfall, for, during some months, the rainfall on the tract may be sufficient without recourse to pumping at all, and, on the other hand, in those places so low that the water cannot drain off by gravity, it may be so much in excess of

FIG. 1.—SMITH AVE. VIADUCT. VIEW LOOKING VERTICALLY DOWN AT EXPANSION AND FIXED SHOES OF 250-FT. SPANS ON ROCKER BENT, SHOWING ABSENCE OF BOLT ON DOWN STREAM SIDE.

FIG. 2.—SMITH AVE. VIADUCT. VIEW LOOKING VERTICALLY DOWN AT FXPANSION AND FIXED SHOES OF 250-FT. SPANS ON ROCKER BENT, SHOWING ABSENCE OF BOLT ON UP-STREAM SIDE.

the needs of the crop as to make it necessary to pump it off. In other cases, the supply cannot be by gravity, but all irrigating water, as well as drainage water, must be pumped from the river, canal or bayou.

The daily rise of the lunar tide on this part of the Gulf Coast is 1.4 ft. When draining for rice, the water table in the ditches needs to be only 1 ft. below the ground surface, when drawn down; for alfalfa, 2 ft.; for sugar cane, 3 ft.; and for garden vegetables and fruits, from 1 to 5 ft.

If the land is intended for rice, it will drain by gravity when the surface of the ground is 1½ ft. above low water, provided the drainage sluices are of the right size, and the rise and fall of the tide is not less than about 1½ ft. For alfalfa, the land must be 2½ ft. above low water; for cane, 3½ ft.; and for vegetables, from 2½ to 5½ ft., according to the variety. Lands below these levels will have to be kept dry by pumping during the cropping season.

Hygrometric Conditions.

Taking rice for example, the months during which water is required for irrigating, in the vicinity of New Orleans, are as shown in Table 1. This table also shows the quantities required, and the average length of time, with the mean rainfall, for the same months, being the mean of eleven circumjacent stations for the last six years (1897–1902).

TABLE 1.—Months in which, in the Vicinity of New Orleans, Water is Required for Rice Irrigation.

Month.	New Method.		Old Method.		Mean precipitation.	Remarks.
	Inches required.	Number of days to be supplied.	Inches required.	Number of days to be supplied.		
March........	0	0	8	10	5.35	Water used in April is put on in March, in old method. For 4 out of the last 6 years there has been less than 2 in. rainfall in May, and for 2 years less than 1 in. in June. The means for New Orleans cover 32 years; for Houma, 12; for Lawrence, 10; Reserve, 1.
April..........	0	(30)	(4)	20	.....	
May...........	0	(31)	0	(30)	2.24	
June..........	4.77	4½	28	25	5.17	
July..........	17.98	18	45	31	6.83	
August.......	9.08	15	15	9	6.19	
Total, including rainfall.	31.83	82½	96	95	25.78	

By Table 1 it appears that, taking the average for the last six years, there has only been one month (May), during the rice growing season, when sufficient water has fallen for the use of the crop by the old method, and but half of the time by the new method. Also, the new requires only 28% as much water as the old method, and requires only 34% as many days of pumping.

TABLE 2.—Evaporation Observations at New Orleans, La.

Authority: United States Signal Service.

Year.	Inches of Evaporation for:					Total for the year.
	April.	May.	June.	July.	August.	
1888	....	....	3.820	9.380	7.960	....
1888	....	....	3.790	....	....	....
1888	3.800	4.200	4.100	....	....	45.400*
1887	....	....	....	4.100	4.300	....
1889	....	....	8.200	8.700	8.900	....
Means	3.800	4.200	4.977	7.393	7.053	27.423†

* Computed for the fiscal year, 1887–88.
† 27.423 in. = total for 5 months' observation (153 days) = 0.179 in. per day.

Now, take the evaporation into consideration. Assuming that the means of the fragmentary observations of the U. S. Signal Service, which are all that are available, at this time, are approximately correct, and considering, also, that it is better to err on the safe side, it appears that the mean evaporation is greater than the mean precipitation in May, July and August, and about equal to it in June, while in April the precipitation is considerably in excess, as shown by Table 3.

The records for New Orleans alone, extending back for 32 years, show a mean annual rainfall of 57.54 in., and the mean for the last 8 years is only 49.63 in., while the mean for the first 8 years (1871 to 1878, inclusive) is 66.98 in. This seems to show cycles of about 20 years, the minimum having occurred in 1891, and the present time being on an ascending node.

Desmond FitzGerald and J. James R. Croes, Past-Presidents, Am. Soc. C. E., and Professor Russell agree that, in the latitude of New England and the Middle States, the evaporation from water surfaces is about equal to the rainfall, taking one year with another. Owing to

FIG. 1.——SMITH AVE. VIADUCT. VIEW LOOKING DIAGONALLY DOWNWARD ON WINDWARD COLUMN CAP, FROM WHICH THE CORNER OF THE 250-FT. SPAN WAS LIFTED, AND THE BROKEN REMNANT OF THE ONLY BOLT HOLDING IT IN PLACE.

FIG. 2.—SMITH AVE. VIADUCT. VIEW LOOKING DIAGONALLY DOWNWARD ON LEEWARD COLUMN CAP FROM WHICH THE SHOE OF THE 250-FT. SPAN SLID OFF.

the greater humidity in the South, it appears to be less. The evaporation for the entire year, at New Orleans, is given by the U. S. Signal Service at 45.40 in. in 1887–88. According to the observation of the New Orleans Sewerage and Water Board, taken for a few months only, it would not exceed 2 in. The Signal Service observations agree more nearly with other authorities.

The quantity of evaporation given in Table 2 is 0.179 in. per day for the 5 months observed. The Signal Service officers, however, state that, in their opinion, their figures should be reduced 20%, making the quantity per day 0.143 in.

General Gillmore estimated, from observations taken on some open ponds in Florida, 0.300 to 0.250 in. per day.

The experiments of the United States Department of Agriculture, at Crowley, La., and in Texas, give a mean of 0.225 in. per day for 67.5 days.*

Evaporation is very largely dependent upon the wind, and this region is completely open to wind and sun. From experiments made by the U. S. Signal Service, it appears that, with the wind blowing with different velocities, the effects shown in Table 4 were produced, as compared with quiet air.

Hydrologic Conditions.

Owing to all these lands being below the level of the Mississippi River, with the drainage away from the river instead of toward it, and the lands themselves being a dead level, there is practically no watershed to be considered. Further, the level of the land is so low that there can be no loss of water by filtration, and the soil is so retentive that very little infiltration need be expected, even in those lands lying below the level of the water outside the protection levees. At least, that is the experience with reclaimed lands in the vicinity. Then, only the effects of rainfall and evaporation have to be considered.

The cultivation of rice requires more water than any other crop, therefore it has been selected for this discussion.

Rice cultivation, on the high, level prairie lands, like those of Crowley, La., and in Texas, and on the low lands, such as those of Georgia and the Carolinas, and those comprised in this belt, is very different as practised in the different localities.

*Bulletin 113, Dept. of Agriculture, Office of Experiment Stations, pp. 28, *et seq.*

TABLE 3.—MEAN MONTHLY AND ANNUAL PRECIPITATION, IN INCHES, IN THE ZONE OF THE DELTA LANDS.

Station.	Period.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Number of Years Covered	Annual Precipitation.
Emilie	1897-1902	4.20	5.15	3.37	4.55	1.89	4.07	5.97	5.96	2.57	3.09	3.51	5.34	5	49.17
Southern University Farm	1897-1902	2.15	6.62	3.34	*5.75	2.38	*4.84	5.59	*4.11	*5.13	*2.86	*3.35	*5.60	3	51.72
Sugar Experiment Station	1897-1902	2.91	5.78	4.14	6.16	1.37	5.52	6.08	4.69	6.22	*2.72	3.40	5.71	5	54.65
Port Eads	1897-1902	3.07	5.58	3.71	5.69	1.47	*3.70	5.69	8.89	8.48	5.63	3.22	5.33	5	60.89
Schriever	1897-1902	3.27	5.65	3.56	5.48	2.69	7.08	7.98	6.57	5.17	3.55	3.27	5.47	6	59.74
Wallace	1897-1902	4.12	5.48	3.62	6.55	1.43	*5.96	6.34	6.90	5.37	2.47	3.96	6.52	5	58.67
Reserve	1901-1902	0.02	*5.18	2.72	6.44	2.52	2.87	7.62	3.58	3.68	2.25	3.89	*6.07	0	46.29
New Orleans	1870-1902	4.56	4.71	5.17	5.20	4.04	6.29	6.27	5.59	4.72	3.08	3.91	4.33	33	57.92
Houma	1890-1902	3.87	5.87	3.08	4.68	2.43	2.17	8.97	6.63	9.95	2.72	2.43	4.06	7½	51.81
Venice	1897-1902	3.24	5.37	3.97	4.08	1.94	6.08	7.62	8.41	3.21	4.41	2.93	4.52	5 to 6	60.78
Lawrence	1892-1902	2.68	6.30	3.88	4.33	2.90	6.88	7.09	6.76	5.44	2.79	2.77	4.03	8 to 10	55.85
Sums	....	34.09	61.04	40.51	58.91	24.61	54.91	75.15	68.09	59.94	35.57	36.64	56.98	....	606.44
	....	....	....	....	....	....	....	....	....	....	....	....	....	....	55.18
Means	....	3.09	5.55	3.68	5.36	2.24	4.99	6.83	6.19	5.45	3.24	3.33	5.18	....	4.51
	....	....	....	....	....	....	....	....	....	....	....	....	....	....	55.18

*Monthly record incomplete.

TABLE 4.—EFFECT OF WIND ON EVAPORATION.

Velocity of wind, in miles per hour.	Evaporation, number of times greater.	Velocity of wind, in miles per hour.	Evaporation, number of times greater.
5	2.2	20	5.6
10	3 8	25	6.1
15	4.9	30	6.3

In Crowley, La., and in Texas, the average length of the season during which water is used is only 68 days, during which time the water is turned on for only 10 days, while in the low lands of Georgia and the Carolinas, it is the custom to supply water at intervals during a season of 144 days, during which time the water is turned on for 95 days, the quantity in the former case being only 27.66 in., as against 96 in. in the latter, the rainfall being included in each case. The quantity of water furnished in the latter case would be only 29 in., if it were not for the fact that during the "harvest flow" the water is changed six times, or every 10 days, to prevent it from becoming stagnant.

The "harvest flow," according to the practice in Georgia and the Carolinas, is kept on for 65 days, steadily, while in Crowley, La., and in Texas, it is only kept on for about 32 days.

These methods may be designated the new and the old practice. The Crowley and Texas method is the new, and the other the old. The former marks a new era in rice growing, and has exploded the old and erroneous ideas that enormous volumes of water are needed to irrigate rice and that evaporation is excessive along the Gulf Coast.

As a matter of fact, numerous observations prove that it is less there than in many other places.

The mean rainfall for July and August is 1.59 in. more on this tract than at Crowley, while the total annual rainfall on this tract is 2.34 in. more, taking the mean of 14 years.

It has been demonstrated conclusively that abundant crops of rice can be raised on the new plan, and, therefore, the writer has adopted it in his estimates of water and pumping.

In the new method no water is put on, from the time of planting for 2 or 3 months, or up to about the middle of June, dependence being placed on the rains to sprout the seed.

It appears from Table 1 that water is supplied to the rice on 32½ days, but part of this is rainfall; so that, leaving out the rainy days, there are only 10 days in the season when pumping is required, in years of normal rainfall. This includes also the quantity lost by evaporation. Therefore, it appears that the pumping of water for irrigation, even if all the water had to be pumped, would not be a matter of very heavy expense, even for rice, and it is less for all the other crops.

Next, consider the length of time during the rice-growing period when siphons can be used. This will depend largely on the location, for, the lower this location is down the river, the less the river is elevated above sea level and the level of the marshes, and, therefore, the less the fall for the siphons and the shorter the time during which they can run.

Table 5 shows the highest and lowest water and the mean elevation, above Mean Gulf Level, of the Mississippi River at New Orleans for 8 years. Also, the number of days in each month that the river was 1 ft. or less above Mean Gulf Level. This table is taken from the hydrographs of the river, made in the office of the State Engineer of Louisiana, from the Bulletins of the U. S. Weather Bureau.

TABLE 5.—Heights of the Mississippi River, at New Orleans, above Mean Gulf Level, for the Years 1890 and 1897 to 1903, Inclusive.

Last day of month.	Lowest.	Highest.	Mean.	No. of days 1 ft. and less.
Oct. 31	0.8	4.9	2.2	1½
Sept. 30	1.0	5.4	2.7	0
Aug. 31	1.7*	9.7	3.9	0
July 31	2.5	13.7	5.5	0
June 30	3.9	16.3	8.4	0
May 31	3.9	17.8	11.7	0
Apr. 30	8.2	18.6	13.9	0
Mar. 31	2.00†	18.6	10.6	0
Nov. 30	0.7	4.2	2.2	10
Dec. 31	0.6	11.4	3.0	7½
Feb. 28	2.1	14.8	7.9	0
Jan. 31	1.3	12.8	5.5	0
Mean	2.4	12.3	6.4	....

* 1900. † 1897.

Table 5 shows that there is not a day during the rice-growing season, from March to September, that the river is not 1 ft. or more above Mean Gulf Level in the vicinity of New Orleans, the lowest water ranging from 1.7 ft. in August to 2.0 ft. in March, and the mean height averaging from 3.9 to 13.9 ft. above Mean Gulf Level.

Theoretically, a siphon should run if the water in the river is only a film higher than that in the marsh, but, in practice, it is found that it is impossible to make joints so tight that some air will not leak in. Also, air is disengaged from the water, and it is stated by prominent engineers who have had experience with siphons in that locality, where they are in common use to take water from the river, as far down as the writer went, or about to Buras, than 1 ft. difference in level is about the practical working limit.

Table 6, taken from the records of the U. S. Mississippi River Commission, shows the number of days, with dates, when the river has been 1 ft. or less above Mean Gulf Level at the Carrolton Gauge, New Orleans, from 1850 to 1902, inclusive.

TABLE 6.—NUMBER OF DAYS WHEN THE MISSISSIPPI RIVER, AT THE CARROLTON GAUGE, HAS BEEN 1 FT. OR LESS ABOVE MEAN GULF LEVEL.

Date.	Lowest.	Date.	Lowest.
Nov. 18, 1850	−0.63	Oct. 24, 1885	+0.77
Nov. 24, 1851	+0.07	Nov. 27, 1886	−0.53
Feb. 4, 1852	+0.47	Nov. 20, 1887	−0.83
Dec. 30, 1854	−0.13	Jan. 10, 1888	−0.83
Nov. 5, 7, 9, 1858	−0.53	Oct. 16–29, 1888	−0.03
Nov. 13, 1859	−0.53	Nov. 9–11, 1889	+0.27
Oct. 18, 1860	−0.83	Oct. 21, 1891	−0.18
Dec 27, 1872	−1.73	Nov. 21, 1892	+0.02
Nov. 30, 1873	−0.21	Dec. 8, 1893	+0.17
Nov. 24, 1874	−0.23	Nov. 15, 1894	−0.18
Nov. 18, 1875	−0.23	Jan. 1, 1895	−0.13
Dec. 30, 1876	−1.83	Nov. 9, 10, 12, 24, 1895	−0.08
Jan. 9, 1877	−1.53	Oct. 9, 11, 1896	+0.62
Oct. 7, 22, 24, 1877	−0.18	Nov. 27, 1897	−0.03
Nov. 22, 1878	−0.23	Jan. 1, 2, 1898	+0.87
Nov. 24, 1879	−0.93	Nov. 4, 1899	−0.13
Nov. 2, 1880	+0.17	Jan. 21, 1900	+0.67
Sept. 10, 1881	+0.17	Dec. 18, 1901	−0.14
Nov. 8, 1882	+0.67	Dec. 21, 1901	
Oct. 15, 1883	+0.37	Mar. 17, 1902	+0.97
Dec. 2, 5, 1884	+0.17		

Table 6 shows that for the last 52 years there have only been 53 days when the river was as low as 1 ft. or less above Mean Gulf Level, or about 1 day per year. It also shows that these low stages have occurred in the months of October, November, December and January, with two exceptions, one of which was in September and the other in February, all being months when no water is required for rice, unless it should be attempted to raise two crops, as is sometimes done. In that case the river records show that in 52 years a total of 30 days might be expected when the water would be too low to siphon in November, 10 days in December and 6 days in January, or, on the average, a little more than ½ day in November every year with a possibility of 10 days, 1 day in December every 5 years with a possibility of 7½ days, and one day in January in every 8½ years.

The study of these records shows conclusively that siphons can be depended upon for irrigation in the vicinity of, or above, New Orleans, the year round, and no pumps will be required. This supposes, of course, that the ditch or irrigating canal is brought up to the levee on the land side, with the water at Gulf Level. The mean rise and fall of the tide at Grand Pass is 1.4 ft. If the canal is fairly straight and unobstructed the rise of the tide at the levee should be about 1 ft., which would give a 5-in. fall at low water, and would give sufficient grade for the canal to discharge on the land at all times from half-tide ebb to half-tide flood, a period of nearly 6 hours, on the average, but variable. Land which is 1 ft. or more above Mean Gulf Level could not be covered at the extreme low-water stage of the river. The grade would be greater at spring tides, as the tide falls lower.

The level of the Gulf is influenced greatly by the winds, and sometimes it falls nearly 2 ft. below the Mean Low-Water plane, owing to a long succession of northerly winds, which lower the water near the shore. On the other hand, it has once risen 6.3 ft. above Mean Gulf Level, or 7.0 ft. above the plane of Mean Low Water, owing to an extraordinary storm. Of course, at such a time the siphons would not work, but, as such high waters always occur during storms and are accompanied by rain, no irrigation water is needed. When the storm subsides the water recedes and the siphons resume their work.

Table 7 shows the monthly means of the highest and lowest water at Ft. Jackson, and the number of days it was down to only 1 ft. or less above Mean Gulf Level, during the rice-growing season of 6 months, March to August, inclusive, for the years 1897 to 1902, inclusive, collated from the Mississippi River Commission's daily gauge readings.

Table 8 shows the extreme high and low water above Mean Gulf Level, at Ft. Jackson, for the whole year, from 1891 to 1902, inclusive, from the reports of the Mississippi River Commission.

TABLE 7.—Monthly Means of Highest and Lowest Water in the Mississippi River at Ft. Jackson, during Six Months of Each Year from 1897 to 1902, Inclusive.

Year.	March.		April.		May.		June.		July.		August.	
	H.	L.	H.	L.	H.	L.	H.	L.	H.	L.	H.	L.
1897..........	5.1	2.5	5.5	5.0	5.5	4.9	4.9	1.3	1.9	0.5	1.5	0.1
1898..........	3.8	1.7	5.0	3.6	4.9	4.0	4.6	2.3	3.0	0.4	2.4	0.3
1899..........	4.8	2.5	5.3	4.5	5.1	3.8	4.1	2.7	3.0	0.9	1.7	0.3
1900..........	4.1	2.1	4.2	3.1	4.0	2.0	3.1	1.2	3.9	0.9	2.3	0.1
1901..........	3.1	0.3	4.1	2.7	4.3	2.2	3.5	1.4	2.5	0.5	5.5	0.0
1902..........	4.5	0.4	4.7	4.0	4.1	1.1	2.4	0.9	2.3	0.9	2.3	0.7
Means........	4.23	1.58	4.80	3.82	4.65	3.00	3.77	1.63	2.77	0.68	2.61	0.25

Number of days the water was as low as 1 ft. or less above Mean Gulf Level; and mean of 5 years.

Possible number of days in 1 year.	14	0	0	0	1	20
Mean number per year.....	3.6	0	0	0	0	10

TABLE 8.—Extreme High and Low Water in the Mississippi River at Ft. Jackson, for 12 Years, from 1891 to 1902, Inclusive.

Year.	Date.	Above Mean Gulf Level, in feet.	Date.	Below Mean Gulf Level, in feet.
1891................................	Mar. 31	5.22	Nov. 30	—1.18
1892................................	June 13	5.17	Nov. 20	—1.18
1893................................	June 15	5.02	Dec. 7	—1.38
1894................................	Apr. 8–9 Oct. 8	4.12 6.32	Nov. 12–14	—1.38
1895................................	Apr. 2–3	2.92	Jan. 9 and Dec. 13	—1.28
1896................................	Apr. 19	4.22	Dec. 4–5	—0.78
1897................................	Apr. 22, 23 Apr. 29 May 14–16	5.53	Dec. 8	—0.83
1898................................	Apr. 23, 26	5.02	Jan. 2 Dec. 26	—0.83 —0.78
1899................................	Apr. 21	5.30	Dec. 4	—0.80
1900................................	Apr. 20–23	4.22	Jan. 14	—0.43
1901................................	May 8, 9, 13	4.32	Nov. 25	—0.98
1902................................	Apr. 14	4.67	Feb. 3	—0.28
Sum................................	23 times	109.04	16 times	15.10
Means................................	....	4.81		—0.94

Table 8 shows the average height of extreme high water at Ft. Jackson to be 4.31 ft. above, and the average height of extreme low water to be 0.94 ft. below Mean Gulf Level. These are the means of the extremes for a term of 12 years, covering the whole of each year.

In explanation of Table 7 it must be understood that at this station two readings of the gauge were taken daily, and this accounts for the apparent discrepancy in some parts of the table, as, for instance, in the July column of low water, which shows a mean for 6 years of only 0.68 ft. above Mean Gulf Level, and yet the lower part of the table shows that in July there were no whole days as low as 1 ft. or less above the datum, with a single exception. The explanation of this is that during a part of the day the water was less than 1 ft. high, but at some time during the 24 hours it was more than 1 ft. high. This is owing to the backing up or raising of the river water by the daily action of the tides. The salt water, being heavier, runs up on the bottom, while the fresh water flows over the top. This tidal influence is felt as far up as Red River Landing, about 225 miles above New Orleans.

The water at New Orleans has been slightly brackish once in a period of about 10 years. At Harvey's, at the head of his canal, the tide rises and falls about 2 in. daily, and in storms about 1 ft., but the water is perfectly fresh. At New Orleans the rise and fall in the river is about the same.

The great Gulf storm of August 14th, 1901, producing the highest rise ever known, raised the river 4.9 ft. at Ft. Jackson, 5.2 ft. at New Orleans, and 1.00 ft. at Red River Landing, above the river height at the time. When the proposed New Orleans and Gulf Ship Canal is built to its full width and depth, the daily tide will probably be about 10 or 12 in. at Harvey's.

Table 7 shows that at Ft. Jackson the greatest height of the water, in the six growing months, for the last 6 years, was 5.5 ft. above Mean Gulf Level, and the greatest mean for any one month was 4.80 ft., for April. The lowest water was 0.3 ft. below Mean Gulf Level in March, and the lowest mean height for 6 years was 0.25 ft. for August. The mean of 6 years' observations shows that in March there are 3.6 days when the water is 1 ft. or less above Mean Gulf Level, and in some years there are 14 days; that in August there are 10 days, with possibly 20; and, once in 6 years, there is 1 day in July.

These conditions, as far down as Ft. Jackson, make the raising of rice by the use of irrigation water obtained from the Mississippi River by siphons, not perfectly satisfactory. The mean of 3.6 days in March, with a possible 14, is not material, as no irrigation water is wanted in March, April or May, provided there is any rain at all, but a loss of water for a mean of 10 days in August, with a possible 20 days, might prove disastrous if it came in the first part of the month, when the water is needed for the "harvest flow."

An examination of the gauge readings for the past 6 years shows that for about three-fourths of that time the scarcity occurred in the last half of the month; the other times occurred intermittently. Therefore, there would not be much danger for the crop, and the writer has observed that rice is grown successfully by the use of siphoned water as far down as Socola. It would be impossible, however, to raise two crops per year as far down as Ft. Jackson, by using siphon water.

Method of Drainage and Irrigation.

The land should be first surveyed and divided into sections of 1 mile square, and these subdivided into 160, 80, 40, and 20-acre lots, to suit purchasers. A protection levee, from 8 to 9 ft. in height above low water, should be built around each section. This is rendered necessary because on one occasion the water of the Gulf, during a severe storm, was raised by the wind 7.0 ft. above low water; and this may occur again. This is said to have been the highest water known since the settlement of the country, and, therefore, the levees should be 8 ft. high in the protected places and 9 ft. high in locations more exposed to the waves. Where the surface of the marsh is 2 ft. above low water, this will make the height of the levee above the ground surface from 6 to 7 ft. The small levees, inside the protection levee, need only be high enough to flood different lots about 1 ft. deep, making the height of these sub-levees from about 1½ to 2 ft. above the levelof the ground.

Each mile section should have a marginal canal or "face ditch" around it on each of its interior sides. This ditch should be 3 ft. wide at the bottom, and about 3 ft. deep, with sloping sides. The 80-acre and smaller lots should be bounded by ditches, 9 in. at the bottom, 3 ft. at the top, and 3 ft. deep.

When the shape of the land will admit, these sections should be laid out in groups, 4 miles wide by 8 miles long, and a protection levee built around each, with a canal for drainage, and another canal longitudinally through the center of the tract. This arrangement will reduce the expense greatly, by dispensing with the high protection levees around each section, as would be necessary if treating one section singly. The arrangement is shown by Fig. 1.

Wooden sluice boxes, with gates, can be laid through these levees, to admit the irrigation water at high tide and permit the drainage water to escape at low tide. This arrangement is feasible in the fresh-water marshes, where the tide rises and falls daily, as the water is not salt. In most places, however, in these lands, the irrigation water will have to be brought in a special canal or ditch from the Mississippi River, or, where that is impracticable, pumped from the bayou.

Each case requires to be made the subject of separate study and treatment, but is perfectly feasible, and each is now being worked at a profit in Louisiana and in the Carolinas and Georgia. The most unfavorable case would be where the irrigation water had to be brought a long distance, in pipes or flumes, and then pumped out after irrigating the land. This might occur in some of the salt-marsh lands, a long distance from the river.

Another case requiring more pumping would occur in the lowest of the fresh-water marshes, where the same pumps would be used to pump in the fresh water for irrigation, and afterward pump it out.

The cost of this pumping, however, would not be as great as might at first be supposed. In the first place, the pumping operations for cane and rice would only extend over from 50 to 70 days, unless it was desired to raise two crops of rice, and, during that time it would not be continuous, by any means. In the case of garden vegetables, it would depend on what was being raised, but, in any event, would not extend beyond the crop-growing period, whatever that was.

In the second place, the lift would be very slight, often not more than 1½ ft., which would permit the use of the cheapest class of pumps, and even wheels alone in many places. Thirdly, the location, for the most part, is a perfectly open and wide expanse of flat marsh, without trees or bushes, except an occasional clump of low trees

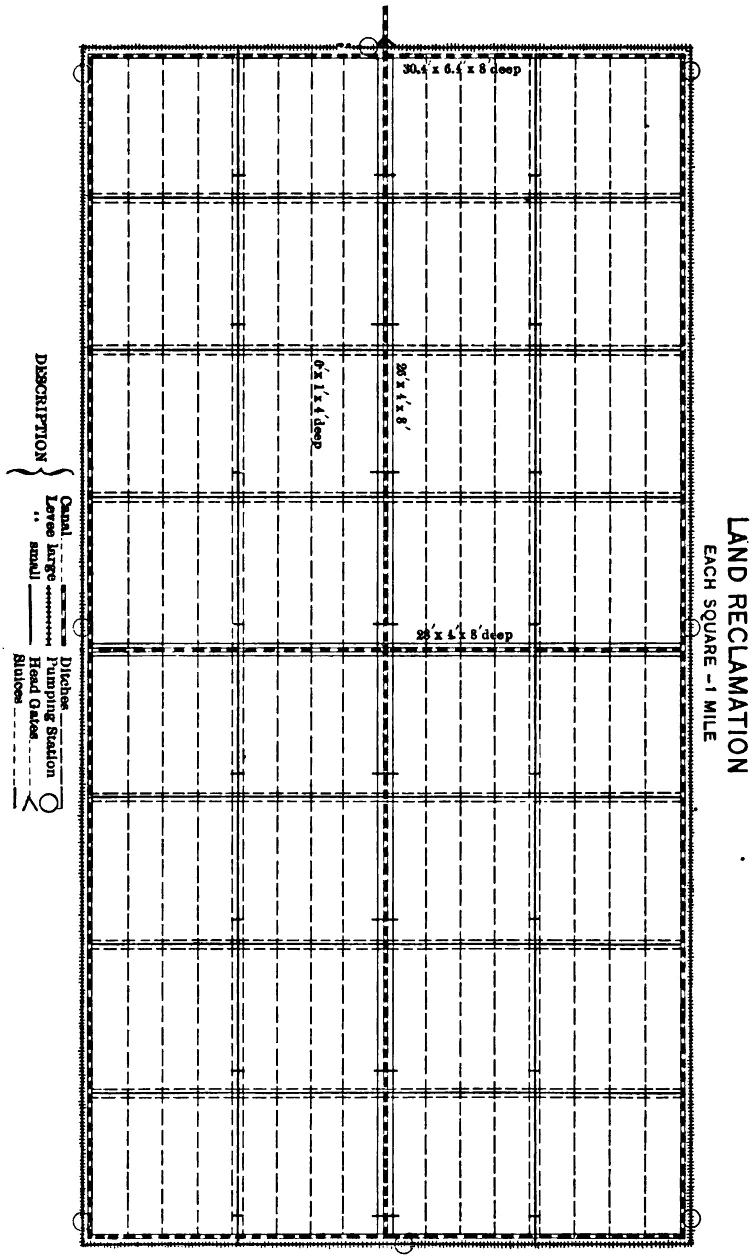
IDEAL PLAN OF
LAND RECLAMATION
EACH SQUARE – 1 MILE
30.4' x 6.4' x 8' deep
6' x 1' x 4' deep
25' x 4' x 8'
28' x 4' x 8' deep
DESCRIPTION
Canal
Levee large
" small
Ditches
Pumping Station
Head Gates
Sluices

Fig. 1.

covering possibly an acre, and a few scattered trees on the borders of the bayous. Therefore, as it is fully exposed to the sweep of the wind from all directions, as much as if at sea, windmills can be used advantageously, as they are on the Zuyder Zee in Holland; indeed, several may be seen on the line of the New Orleans, Ft. Jackson and Grand Isle Railroad, which runs through this territory. The cost of pumping by windmills is very low, consisting chiefly in repairs to the mill and pump, and the daily oversight by an oiler, who could be the watchman tending the sluices, embankments, etc.

The cost of oil and repairs would not be more than 10% of the cost of the mill, which may be taken at $300 for the larger sizes, set up.

An objection to windmills, besides the uncertain character of their power, is the small size and small duty of the commercial mills found in the United States.

One of these mills, 16 ft. in diameter, with the wind at a velocity of 20 miles per hour, and with a lift of 30 ft., will pump 4 224 gal. per hr., which is only about one-fifth of 1% of the capacity of a 42 by 16-in. Menge pump, and one of the latter will be required to irrigate every 320 acres, provided a case should occur where all the water has to be pumped to the fields. It would require about 473 of these windmill pumps to do the same work, or one pump to 0.68 acre, that is, provided all the water of irrigation has to be pumped, which would hardly ever be the case; but it serves to show the comparative duty of the windmill and electric or steam pump.

The experiments made under the auspices of the United States Geological Survey* show tests made of nearly all the windmills on the market in the United States. According to these tests, the 16-ft. "aermotor" appears to be among the best, and this never exceeds 2 h.p., with a 20-mile wind, and with a 15-mile wind may be assumed to be only 0.8 h.p.

The old-fashioned Dutch windmills, with a diameter of 70 ft., yield 7 h.p., with the same wind, according to the experiments of Coulomb in Holland. This would equal one mill for about every 5.9 acres, for irrigating, on the same basis as computed previously, and the ratio of the Menge pump to these windmills would be about 1 to 54.

The cost of one of these large Dutch mills would not fall much short of $800, therefore, windmills would be vastly more expensive

* "Water Supply and Irrigation Papers, Nos. 41 and 42."

than electric pumping, and there would be some uncertainty as to their action. However, for use in small separate fields, or as an auxiliary, they would be economical. In cases where the irrigation water is supplied by gravity, and the pumps are only required to take off the surplus water, one 16-ft. aermotor will de-water 1.38 acres, and one Dutch mill about 11.8 acres per day.

Mr. J. B. Watkins, reporting on the methods pursued in reclaiming large areas of tide marshes in Louisiana, says:*

"Our plan of reclamation is to build dikes, along the Gulf, rivers, lakes and bayous, of sufficient height and strength to prevent overflow of each in the event of floods from rain and storm tides, and in this we will be materially assisted by the natural levees found in many places along these waters.

"We cut, parallel to each other, and ½ mile apart, canals 18 ft. wide and 6 ft. deep. At right angles with these, at intervals of 2½ miles, we cut larger canals, thus forming the land into oblong blocks, ½ mile by 2½ miles, each containing 800 acres. Across these blocks, at proper intervals, we cut lateral ditches, 30 in. deep by 8 in. wide at the bottom, flared to 30 in. wide at the top.

"These canals are cut, the levees formed, and the dikes are, to a considerable extent, built by the use of powerful floating steam dredges. The smaller ditches are cut by ditchers propelled by steam power, passing through but once, at the rate of 1½ miles per hour. At the proper localities, we erect automatic flood gates, by means of which we control the stage of water in the canals, and the necessary volume of water is regulated to some extent by the ebb and flow of the tide. This is supplemented by the use of powerful wind pumps, and when the natural elements will not accomplish the work, we readily move upon the canals to the spot our ditching, plowing and cultivating engines and attach them to pumps. Thus arranged, with control of the water, these blocks of land are in condition for the most successful rice culture.

"Rice may be planted any time from February to June, very much the same as wheat, and upon ground similarly prepared. When it has reached a growth 2 in. high, water is let in upon it and the ground gradually flooded; care being taken not to cover any of the plants with the water. The land is kept flooded sufficiently to kill all the grass and weeds, until the rice is about 18 in. high. It has then sufficient start to choke down any foreign growth, and the water may be drawn off and the ground allowed to become dry and firm for harvest time, which may extend over several months, according to the

* "Tide Marshes of the United States," Special Report No. 7, U. S. Department of Agriculture.

time the seed is sown. Rice is harvested and threshed with the same kind of machinery as used for wheat."

His dredges have a capacity of a mile of canal, 6 by 18 ft. per month each, and he plowed 70 acres of land per day with gang plows drawn by traction engines.

In proportioning the canals for supplying irrigation water, the quantity of water required is determined by previous experience in rice growing. This must be increased by the quantity lost by evaporation, both in the field and supply canal, and this, in turn, must be diminished by the average recorded rainfall in that vicinity. In a tract as large as this, extending fully 100 miles north and south, and east and west, the hygrometric conditions will vary, and the computations adaptable to a plat at the west might differ considerably from those for one at the east end. Again, if the supply canal is long, more water will be lost by evaporation than if it is short. Therefore, each case must be made the subject of special study.

From experiments conducted by the U. S. Department of Agriculture, at Crowley, La., it appears that the depth of water required on the rice field during a season of 64 days, during which time the water was turned on for 12 days, was 26.51 in., of which 10.04 in. were rainfall; and 14.47 in. of this were taken up by evaporation, leaving the net depth of water received by the land 12.04 in.

At Raywood, Texas, similar experiments by the same parties gave 28.81 in. of water required by the land, of which 19.66 in. were supplied by the pumps, and 9.15 in. were rainfall. Of this, 16.03 in. were taken up by evaporation, and the net depth of water received by the land was 12.78 in., extending over a period of 71 days, during which the water was turned on for 18 days. These two experiments show a very close coincidence, and furnish valuable data for computing the water required in similar locations.

Where the water has to be brought a long distance, from the river or Harvey's Canal, reservoirs can be easily made, in many cases, and these will greatly reduce the length of the pipe or flume, and often cause it to be dispensed with entirely. In making these reservoirs, advantage would be taken of one of the many small bayous or creeks, of which there are a great many running through the marshes in all directions. These would be dammed at the point nearest the land to be watered, and the salt water, if any, pumped out. A ditch or flume would then be built at the upper end to bring in the fresh water, and the creek or bayou would form the reservoir. It would be necessary to run protection levees along both banks to keep out storm water.

Accurate topographic and hydrographic charts of these marshes are necessary in order to locate and plan these reservoirs and the system of levees and canals.

Methods and Cost of Pumping.

The cost of running the pumps will vary chiefly according to the fuel used, as colored men, working for very low wages, can be found who are capable of attending to them, as is done now. The U. S. Department of Agriculture has made tests of, and has reported on, the cost of fuel per acre irrigated in Louisiana. The following is quoted from this report:

"Three kinds of fuel are used to make steam for irrigation pumps in the rice district: coal, wood and oil. Coal is the most expensive because of the long hauls necessary, and oil, based upon the experience of the one year that has passed since the Beaumont oil basin was discovered, is by far the cheapest and most satisfactory. Pittsburg, Kans., bituminous coal sold as high as $4.75 per ton,* and wood at $1.50 and $3.00 per cord, while the oil delivered f. o. b. in car lots, cost from 48 to 62½ cents per barrel. Based upon reports received, the cost of fuel, necessary to irrigate an acre of rice, was between 60 cents and $1.00 when oil was used, between $2 and $3 per acre when wood was used, and fully as much for coal as for wood. Crude mineral oil has proven a most satisfactory fuel. A uniform and high pressure in the boilers is easily maintained, and one fireman can easily handle a battery of half a dozen or more large boilers. The combustion is practically complete, and no injury to the boilers from the hot blast has yet been noted."

Fuel oil is now 75 cents per bbl. (42 gal.) in tanks on cars at New Orleans, and 12½ cents per bbl. for less than 10 000 gal. per month, delivered in the consumers' tank, by pumps, or 9 cents per bbl. for quantities of more than 20 000 gal. per month, the latter being brought in tank ships from Texas.

Two styles of pumps are commonly used about Crowley for irrigating, the rotary and the centrifugal. The former gives more efficiency, but it is heavy and requires very solid foundations as it is geared directly to the engine shaft, and therefore no settlement is allowable. The centrifugal, on the contrary, is generally run with a belt, and considerable settlement will not derange it. It is light, and

*This was at Crowley, La., in 1901. It is now (1903) $4.25 a ton, delivered on lighters, in New Orleans; Alabama coal, $3.

easy to keep in repair. A variety of Archimedean screw, made in New Orleans, called the Menge pump, after the inventor and maker, is probably the best for the low lifts that prevail on this land.

Rainfall and Infiltration.

In computations based on rainfall, it is of the first importance to have as many stations as possible in the tract or around it, as the precipitation varies greatly even in as comparatively small an area as that of the City of New Orleans. In this case, fortunately, there are numerous stations almost surrounding the tract, so that a very fair general average of the rainfall can be obtained. The monthly precipitation at each station for a period of 7 years shows that the mean precipitation on this tract varies from 2.24 in. in May to 6.83 in. in July.

On part of this tract the water that is put on for irrigation must be pumped off, minus what has been dissipated in evaporation and absorption. All this soil is so retentive and the grade is so low that there will be no loss by filtration, or seepage. The gain by infiltration will be very slight, judging by the experience of the planters and those living, or cultivating, immediately back of the river levees. This is owing to the very retentive, puddle-like character of the soil.

The late John M. Goodwin, M. Am. Soc. C. E., one of the commissioners of the Pennsylvania Ship Canal, estimated the infiltration and leakage in that canal as 20% of the entire volume. Of this, the leakage would probably constitute about 75%, leaving only 25% for infiltration proper, or 5% of the entire volume of water. The experiments reported by General Gillmore, of filtration in Florida canals, give a very much greater quantity than would be expected on this land, owing to the sandy and extremely porous character of the Florida soil. In the absence of any known experiments in this line in this neighborhood, the quantity can only be approximated, but it is safe to assume that it will not be large.

An analysis of the rainfall from 1872 to date and Table 3 show that the mean annual precipitation is 55.13 in.; that there is no well-defined rainy or dry season; that the mean monthly precipitation is 4.51 in.; that the greatest monthly precipitation, in any one year during the last 6 years, at any one of the surrounding stations, was 19.55 in.; that the mean of the greatest and least monthly precipitation for each year for 6 years is 10.77 and 0.79 in., respectively.

It also appears by the table of observations taken by the New Orleans Drainage Board, that from 1881 to 1894, inclusive, there occurred one rainstorm of 3.60 in. in 1 hour, and several others of shorter duration, but nearly equal intensity.

From the "Table of Excessive Precipitation" furnished by the Weather Bureau, and extending from 1870 to date, it appears that there has been as great a fall as 9.22 in. in 2 days, and 7.40 in. in 1 day.

Table 9 shows the months and days of the month when excessive precipitation has occurred during the last 32 years, arranged to show the months of greatest excessive precipitation. This table has been collated from the records of the U. S. Weather Bureau and the observations of the New Orleans Drainage Board, at New Orleans, Lawrence, Houma and Venice.

From Table 9 it appears that the months when short falls of great precipitation occur most often are, in the order named: April, September, February, March, August, June, July, December, October, May, November and January. Of the six most pronounced months, four are months of rice growing. Therefore, there may be an occasional engorgement of the drainage system during these months, for a short period. The flooding of rice for three or four days, or even several weeks, at some periods of growth, will not prove harmful.

The most excessive precipitation of this kind occurs in April, when twenty-four storms, each, with a mean precipitation of 4.02 in., have occurred in the last 32 years. These storms may last one or two, and, in rare instances, three days. This analysis shows that there is not much to fear from these short storms of excessive rainfall.. The four months of most excessive precipitation are months when no water is put on the rice, dependence being had entirely on the rainfall, which is generally sufficient. If there should be a drought, the water would have to be turned on; therefore, when these rainfalls occur in March, April or May, they will be an advantage. Those that occur in June, July or August would cause an engorgement if they happened to come just after the water had been let on, and might necessitate starting up the pumps, to relieve the land of the surplus. On the other hand, if they occurred just before the water was due to be let on, they would save the trouble of opening the gates or the cost of pumping so much water, in fields where pumping was required for

TABLE 9.—EXCESSIVE PRECIPITATION, IN INCHES, AT POINTS NEAR NEW ORLEANS, LA., DURING THE LAST 32 YEARS.

Day.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1			3.02									
2							3.49		4.66 3.25 3.73	2.75 2.95	2.61	
3	2.82			6.01 3.80		2.90	2.48 3.25		1.24 3.40		3.23	
4						2.70	7.52				2.61	3.82
5	2.62			2.96	2.00 3.98		3.38					
6						3.02 2.00 2.86	1.40				2.99	
7				5.51	4.09		1.00 2.66	1.62			4.39	
8	4.07		3.36	9.22								
9			2.73			2.68		4.30				
10	2.72 3.12 2.85					3.50 3.68			3.01 5.27	3.47	2.70	
11		2.60 2.72	2.75					3.10 2.77	3.08 2.77	4.15		
12		3.14					3.00		9.30		2.80	1.26 2.85
13							1.15	2.58 2.60	3.97			3.40
14		3.68	3.95 4.04 3.11	3.25				4.14	1.20		2.32	
15	4.28	2.60 4.02	3.66				1.25	3.70 2.67	2.88			2.92
16		5.10		2.82 3.84	3.89		3.11	3.85		2.68		
17		3.00 5.76 2.78		1.70 4.00 3.98					2.52 7.22			
18				5.92 2.68		2.70	2.01	4.25				2.54
19	3.71	5.25 5.71	3.98	2.88	3.54	1.04	4.07			3.19		
20		2.60 3.59		2.90	3.29			8.90	2.78			
21		2.72					2.74	4.08	3.50	2.55		
22				7.49		1.25				2.00		
23			3.60	4.84 3.59		3.08		1.25		4.13		
24		4.21	2.69 2.60 4.50	2.18 2.05 7.40	4.56				3.01			
25				1.35			2.61	2.80	1.39 3.60 5.28	2.54		
26	2.71					2.86 4.44			3.23		3.35	
27			3.72		4.49			2.84				

TABLE 9—(*Continued*).

Day	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept	Oct.	Nov.	Dec.
28..........	......	......	......	2.72	......	......	......	......	2.88	2.66	......	......
	......	......	......	......	......	......	......	......	2.85	......	......	3.27
29..........	......	......	......	......	......	......	......	......	......	......	......	2.90
	......	......	......	......	......	......	......	......	......	......	2.65	......
	......	......	3.61	......	......	3.20	......	......	......	3.52	2.52	3.40
30..........	......	......	......	......	......	5.48	......	......	......	......	......	......
	......	......	......	3.58	5.10	2.76	......	......	5.60	3.04	......	......
31..........	......	......	......	......	......	......	......	......	......	......	......	11.35
	......	......	......	......	......	......	......	......	......	......	......	2.70
	......	......	......	......	......	......	......	......	......	......	......	2.59
Sum.........	28.00	59.43	56.52	96.57	34.44	50.15	45.12	55.45	91.57	39.58	32.17	43.00
Number of storms......	9	16	15	24	9	17	16	16	25	13	11	12
Mean precipitation.. ...	3.21	3.71	3.77	4.02	3.83	2.95	2.82	3.46	3.66	3.04	2.92	3.58

irrigation. As these storms are not so violent, or rather the precipitation is not so great, in June, July and August, the chances are rather more than even that they will prove a benefit, rather than a drawback, the mean precipitation only ranging from 2.82 to 3.46 in. for each storm, of which only sixteen or seventeen have occurred in 32 years, or one every other year in these months.

The rainfall, therefore, to be pumped off the land after it has served its purpose, is the mean of the monthly precipitation occurring during the rice-growing months, as shown in Table 11, plus the quantity put on the land monthly by siphons or pumping, plus the quantity added by infiltration, and minus the quantity lost by evaporation; and the pumps must be capable of taking this water off in a short time, say two or three days.

Table 10 shows the greatest monthly precipitation in each year, for the last 6 years, in the months from March to August, inclusive, in each of the eleven stations surrounding this tract.

Part of Table 10 is collated from the record of rainfall at New Orleans kept by the U. S. Weather Bureau, and part from the record of rainfall kept by the New Orleans Drainage Commission.*

These two records are interesting as showing the difference between two records kept on the same dates in different parts of the same city, and they emphasize the fact that the records of one station cannot be accepted as a basis for computations, but that the means of all cir-

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cumjacent stations must be used to represent the actual precipitation on any tract of more than 1 mile square. They also make plain the absurdity of carrying out such computations to more than two places of decimals.

TABLE 10.—Greatest Monthly Precipitation, in Inches, in the Months from March to August, Inclusive, for the Past 6 Years, at Eleven Stations.

	Mar.	Apr.	May.	June.	July.	Aug.	Remarks.
	6.54	12.78	8.14	7.19	10.79	8.66	(Greatest in 32 years for New Orleans, the other for 6 yrs.)
	5.80	11.70	6.29	9.10	10.82	5.70	
	6.78	5.75	4.58	7.80	12.38	10.67	
	6.50	10.69	8.85	10.91	10.71	8.05	
	6.84	8.18	6.07	8.09	18.39	9.69	....
	6.09	12.07	4.30	7.78	14.30	8.50	....
	8.85	18.68	2.70	17.61	8.82	14.74	....
	11.82	18.78	18.68	8.01	9.87	22.74	....
	5.22	10.64	10.38	18.33	9.04	7.58	....
	.....	.....	.....	.....	8.75	.....	....
	.....	14.20	.....	8.04	6.20	.....	....
	.....	.....	.....	.....	8.96	.....	....
	.....	12.27	.....	12.05	10.71	.....	....
	.....	.....	.....	.....	18.39	.....	....
	.....	.....	.....	10.47	12.38	.....	....
	.....	.....	.....	.....	11.51	.....	....
	.....	.....	.....	4.70	9.61	.....	....
Mean.	6.49	11.43	6.65	9.69	11.27	10.70	

Water Required.

Table 11, compiled from the records of the U. S. Weather Bureau, shows the monthly mean precipitation and the greatest monthly mean precipitation for each of the six rice-growing months, with the difference between them, and the quantity required for rice cultivation. This table shows that once every 6 years, on the average, there will be an excess of rainfall above the means for that month, and these months of larger precipitation may occur in groups or all in one year.

The mean excess, as shown in Table 11, amounts to 48 per cent. Table 11 shows that in some months of excessive rainfall, when water is required for rice, 4.92 in. more water falls than is required, which is 31.83 in., including what will be lost in evaporation, and adding 5% for infiltration. As the quantity lost by evaporation here is 4.17 in. more than the mean of Crowley and Texas, the quantity of water required here is increased that much more than the requirements in those places.

TABLE 11.—GREATEST MONTHLY MEAN PRECIPITATION, IN INCHES, FOR EACH OF THE SIX RICE-GROWING MONTHS, ETC.

Month.	Greatest monthly.	Monthly.	Difference.	Inches required.	Remarks.
March	6.49	3.68	2.81	Very little	(Means of 11 circumjacent stations for 6 years.)
April	11.43	5.35	6.08	" "	
May	6.65	2.24	4.41	" "	
Sum	24.57	11.27	13.30	0	
June	9.69	5.17	4.52	4.77	
July	11.27	6.83	4.44	17.98	
August	10.70	6.19	4.51	9.08	
Sums	31.66	18.19	13.47	31.83	
Means	9.37	4.91	4.46		

From the foregoing, Table 12 has been compiled, and shows the quantity required for the land and which is supplied by rainfall, irrigation and seepage; also, the quantity lost by evaporation, leaving the remainder to be pumped off in those parts of this tract too low to drain by gravity at low tide.

TABLE 12.

Month.	Inches of Water Supplied by:			Inches of Water.		
	Rainfall.	Seepage.	Irrigation.	Required.	Lost by evaporation.	To be pumped off.
March	3.68	0.18	0	3.86	3.79	0.07
April	5.35	0.27	0	5.62	3.80	1.82
May	2.94	0.11	0	2.85	4.20	0.0
June	5.17	0.26	0	5.43	4.98	0.45
July	6.83	0.34	10.81	17.98	7.39	10.59
August	6.19	0.31	2.58	9.08	7.05	2.03
Totals	29.46	1.47	13.39	43.66	31.21	14.96

Table 12 shows that, generally, the rainfall and seepage or infiltration, in the vicinity of New Orleans, is enough, or more than enough, for the first four months of the rice crop, but that water must be supplied by irrigation in July and August, the quantity to be pumped off afterward being about equal to that supplied by irrigation in these months. In March, April and June the evaporation is nearly equal to the rainfall, and in May nearly double. Comparing the evaporation with the quantity required and the greatest monthly precipitation, as given in Table 11, it will be seen that in months of greatest precipitation more than enough rain falls every month, except in July, when 6.37 in. have to be supplied by irrigation.

The greatest quantity to be pumped off in any one month is 10.55 in. in July. This would be equivalent to $43\,560 \times 0.88 \times 7.48 = 286\,729$ gal. per acre, and one Menge pump of the largest size would require 15 days to take this water off 4 sections, and 4 pumps would do it in 3¾ days, which is sufficiently rapid. The greater part of this land, however, is elevated 1 ft. or more above the Mean Gulf Level, and this would be sufficient to drain off by gravity, and the pumps would only be required to lower the water in the ditches. The pumps used for irrigating, in those places where siphons could not be used, could be used for drainage by a simple arrangement of valves and a by-pass.

The Menge pump may be described as an Archimedean screw, or perhaps as a turbine wheel, set in the water and run in the opposite direction to a wheel for power, or to a kind of submerged centrifugal pump, set with the shaft vertical in a square wooden box, and run with a belt from the engine. These pumps are used extensively in the vicinity of New Orleans for low lifts, and are much cheaper than even the centrifugal pump, which is cheaper than any of the others. The largest size Menge pump, 42 by 16 in., has a stated capacity of 2 000 000 gal. per hr. with a lift of 10 ft., or 33 333 gal. per minute, which equals 147.31 acre-feet per 24 hours, equal to 5 155.3 acres irrigated in 70 days, therefore eight of these pumps would be sufficient to irrigate four sections (allowing for land taken up by levees) 6 in. deep in 1 day, or the whole tract in 8 days, supposing the evaporation to equal the rainfall.

The price of a pump of this size is $937.50.

This computation is based on the rainfall and evaporation being the same for this land as at Crowley. This is not exactly correct, but it is not very different.

As stated before, every locality requires a separate study and computation based on the precipitation, evaporation, length of supply canal, etc., at that particular place.

The writer believes that the most economical plan will be to run all these pumps by electricity, an electric motor being placed in each station and connected with a central power-house. This will give a power ready to be used at a moment's notice; dispense with all the pump engineers and firemen except one each at the central station, who will run the whole plant; will save the expense of installing so many boilers and engines; and the expense of separate oil tanks and pipe line to each pumping station, or the greater expense of transporting fuel by lighters to these stations.

There is so little wood on this tract that this would be an expensive fuel.

Cost of Reclamation.

The cost of reclaiming the swamp lands, open and without trees or bushes, and supposing them to be laid out in 1-mile sections of 640 acres, and these sections grouped into one tract of 8 by 4 miles, will be as follows:

A protection levee, say 8 ft. high, with 4 ft. crown and 28 ft. base, and slopes of 1½ to 1, will be required all around the tract. This will be thrown up on the outside of the canal from which it is taken, and its size will determine the size of the canal, an allowance of 15% being made for shrinkage, which gives, for the size of the canal, 30.4 ft. wide on the top, 6.4 ft. wide at the bottom, 8 ft. deep, and with slopes of 1½ to 1. A head-gate will be built to connect this canal with the nearest navigable bayou or creek, to allow of its being navigated by boats.

A canal of smaller size will be built up along the center of the tract longitudinally and also one transversely, to allow of convenient access to different parts of the tract by boats. Smaller canals will be dug around each section and through the center, north and south, with ditches on intermediate lines, as shown by Fig. 1. The actual cost, if laid out on this plan, will be as given in Table 13.

If this land should be taken in single sections, and it should be necessary to bring the water in a long flume or ditch, and pump also, with a reservoir, the cost would amount to $16.50 per acre, while, on the other hand, in those locations where no pumping would be necessary, but the land could be flooded and drained by the action of the tide, as previously explained, the cost would be only $2.41 per acre. Each of these cases can occur on these lands.

TABLE 13.

Excavation, made into levees, 631 381.2 cu. yd. at $0.05	$31 569.06
42 by 16-in. Menge pumps, eight at $937	7 496.00
Eight pump houses, and setting up pumps	4 000.00
Electric motors (50 h.p.), eight at $500	4 000.00
Feed wire, No. 0 copper, say 25 miles	5 930.00
Poles, insulators and setting	1 500.00
Sluices set, 192 at $30	5 760.00
Two wooden head-gates, set up	4 000.00
Plowing, and burning levees	1 000.00
Surveying land and dividing into sections, 52 miles at $6	312.00
	$66 567.06
Contingencies, omissions and engineering, 10%	6 656.71
Total: 20 480 acres at $4.35 per acre	$73 223.77

As to the time required to accomplish the work in the case under consideration, which may be taken as a fair average condition, if the latest and largest improved suction dredges are used to dig the canal and make the levees, which they can do by using a plank form, which would be mounted on broad wheels and dragged along by using a snatch block and tackle attached to the engine of the dredge, all the canals of large size could be dug in a month after the machinery was on the ground, and the smaller canals and ditches in 4½ months, some of the latter being cut at the rate of 1 mile in 1½ days. Allowing for the time required to plow the levee bases, and for delays and stoppages incident to such work, it is safe to say that the 20 000 acres could be ditched and leveed ready for a crop of rice, with all gates set, in 9 months.

The cost of operating this plantation, outside of the farming operations, which would be done by the lessees, would be very little indeed, and would only be the cost of fuel and the wages of three or four pump men, and the cost of running an electric or alco-vapor launch, to be used by these men in visiting the pumps for the purpose of oiling and repairing them, and the windmills, etc., opening and closing the water gates and keeping the canals clear of weeds, total per year $20 000.

As to the income, the opinion is unanimous, among all in that district with whom the writer has conversed, that these reclaimed lands would rent easily and quickly for $5 per acre, up to $14, with water supplied, paid in rice.

TABLE 14.

20 480 acres at $5 per year		$102 400.00
Less cost of superintendence, fuel and operation	$20 000.00	
" commissions and advertising	10 000.00	
" interest on first cost, 10%	8 906.38	
		38 906.38
Net profit per year, $3.10 per acre		$63 493.62
Or, if planted in sugar, 50 cents per ton of cane raised, or $10 per acre		$204 800.00
Less cost of superintendence, interest, etc., as before		38 906.38
Net profit per year, $8.10 per acre		$165 893.62

If the land were planted with rice by a company, and the rice were sold to the millers, then, allowing a profit of 20% from the farming operations, which is certainly low, and taking the average production per acre at 14.2 bbl. for this region (the mean of several statements), the account would stand thus:

20 000 acres, at $14.2 bbl. per acre = 284 000 bbl. of rough rice at $3.50	$994 000
Net profit, 20% per year	198 800

As rice is not a cultivated crop, only requiring planting, watering and harvesting, and no fertilizers, the net profit thereon is much greater than on wheat or most other crops, and is nearer 40 than 20 per cent. It is highly probable, also, that these lands will produce as much, if not more, than those of Georgia and the Carolinas. Dr. Knapp states that the profit on rice growing is 55 per cent.*

The reports of the U. S. Department of Agriculture state that when the first large canal near Crowley, La., was completed and

* "Rice Culture in the United States," Farmer's Bulletin No. 110, U. S. Department of Agriculture.

operated successfully, the average price of rice lands rose rapidly from \$7 to \$10, \$15, and \$20 per acre.* Rice lands under the large canals around Crowley, La., are now held at an average price of \$30 per acre, a few choice locations bringing \$50 per acre in 1901.

These delta lands are so much richer than the prairie lands that, measured by their productiveness, they should easily bring twice as much, to say nothing of their immediate proximity to the metropolis of the State.

These lands are of the same fertile character as the rice lands of the Carolinas and Georgia. The former have produced 32 bbl. of rice per acre and more than 8 000 lb. of sugar. The prairie lands of Louisiana and Texas do not produce an average of more than 12 bbl. per acre.

These delta lands, as soon as they are relieved of water, can be leased to small farmers for from \$5 to \$6 per acre per year, or for 4 bbl. of rice where irrigation water is furnished and for 2 bbl. when the land is only drained. In some cases the leases, on such lands in this district as have been reclaimed, are for one-quarter of the crop, and, in this region, there is great demand for these reclaimed lands.

There is no doubt that they would be worth \$200 to \$300 per acre for truck farming when reclaimed, as some of these lands are only 3 miles from the center of the City of New Orleans, with a population of about 340 000.

* "Irrigation of Rice in the United States," 1902, Bulletin No. 113, p. 14.

Vol. XXX. NOVEMBER, 1904. No. 9.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE STRUCTURAL DESIGN OF BUILDINGS.

Discussion.*

By Messrs. W. B. W. Howe, Charles Worthington, J. R. Worcester, Joseph H. O'Brien, Henry B. Seaman, Augustus Smith, R. D. Coombs, Jr., F. T. Llewellyn, Theodore Cooper, Henry W. Post, Gunvald Aus, and J. K. Freitag.

W. B. W. Howe, M. Am. Soc. C. E. (by letter).—This paper is valuable in bringing together and contrasting the practice of various cities in the matter of permissible loads and stresses. There seems to be no good reason why some of these differences should exist, and it is to be hoped the discussion will lead to greater uniformity in this respect. Mr. Howe.

In the matter of determining the bearing power of piles, it does not seem wise to incorporate in a general specification a requirement based upon a formula of uncertain practical value, particularly as the results given by it are confessedly not to be relied upon beyond certain fixed maximum limits.

Long ago the writer reached the conclusion that the actual safe bearing capacity of a pile could not be determined by a general formula, and tests made by him from time to time in different localities seem to bear out the correctness of this conclusion.

If a pile sustained its load by transmitting it entirely to its point, there might be some reason to expect a formula based upon the settle-

*This discussion (of the paper by C. C. Schneider, M. Am. Soc. C. E., printed in *Proceedings* for September, 1904), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.
Communications on this subject received prior to December 24th, 1904, will be published subsequently.

Mr. Howe. ment under the last blow of the hammer to yield approximately correct results, but, as is well known, under certain conditions, nearly the whole supporting power is derived from the friction of the material penetrated upon the sides of the pile, and this friction is not developed until after a considerable period of rest. This being the case, the settlement at the last blow, under such conditions, is not the controlling factor in the ultimate result.

From a set of tests made by the writer some years ago, where the piles were driven in deep alluvial soil, it was found that with a settlement of from 12 to 15 in. at the last blow of a 2 000-lb. hammer falling 20 ft., the pile could be safely loaded to about 200 lb. per sq. ft. of surface of contact, and, in one instance, where the settlement at the last blow was as much as 26 in., the pile carried, without further settlement during the time it was under observation, a live load of 15 000 lb. In this instance the average diameter of the pile was 10 in., and the depth in the ground 40 ft.

Upon another occasion, a pile trestle, in which the piles were driven to a final penetration of 1½ in., the weight of the hammer and the fall being similar, settled badly under the same loading, and continued to settle until the track was taken off, the material penetrated being different in the two cases.

The writer does not believe his experience to have been at all unusual. The method he uses is to test the individual foundation by actually driving test piles where some positive information cannot be obtained in any other way, and he thinks this the only safe method.

Mr. Worthington. CHARLES WORTHINGTON, M. AM. SOC. C. E. (by letter).—In the writer's opinion, the specifications submitted in this paper are rational and complete, and in them Mr. Schneider has presented a most valuable instrument to those having work of this kind to design.

On the subject of wind pressure: The writer, a while ago, observed the erection of two buildings just across the street from each other, each about sixteen stories high. The framework in one appeared to be thoroughly braced for wind stresses, by deep girders and gussets; the other, apparently, was designed as though there was to be no wind in the vicinity. These buildings have been completed several years, and used for office purposes; one appears to be as good as the other, and it would probably be hard to convince the owners of the lighter structure that they should have spent more money in the steel frame of their building, should have paid more for something hidden entirely from view and apparently unnecessary.

The writer does not mean by this to advocate designing structures of this type without regard to wind forces, but he does believe that there are elements of strength here which are not always considered.

The very large direct loads producing practical continuity in the columns; the accumulated portal effect of many girders properly connected to the columns at top and bottom; a well-chosen system of deep girders, with columns properly located for resisting bending stress; and the great resistance offered by the substantial outside walls, all combine to resist effectually, in most cases, all wind stresses, without requiring any actual increase in the cost of the framework, excepting in gussets or lug angles on girders, and extra rivets for connecting them to the columns. Mr. Worthington.

The writer thinks that, for completeness, the following clause ought to be included.

a.—Rivets in tension members shall be located so as to cut out as little of the section as possible, the rupture of a riveted tension member being considered as equally probable, either through a transverse line of rivet holes or through a diagonal line of rivet holes where the net section does not exceed by 30% the net section along the transverse line. (From Cooper's specifications, modified.)

b.—Tension and compression members shall have connecting rivets placed, as far as possible, on the line passing through the center of gravity of the section, or else grouped symmetrically about the same.

c.—In locating rivets of a group intended to transfer a certain stress, they shall be placed so that the axis of stress, if extended, will pass through the center of gravity of the group of rivets.

d.—Rivets connecting flange angles of girders to the web shall be spaced, at any point in the length of girder, not farther apart (in inches) than the product of the distance (in inches) between the rivet lines in the top and bottom flanges, by the governing value of one rivet (in shear or bearing, as the case may be), divided by the shear at that point.

The writer also favors using for compression a flat unit stress of 12 500 lb. for values of $\frac{l}{r}$ equal to or less than 60, instead of the variable unit stress given by Mr. Schneider. This simplifies the calculations, and, for such short columns as are ordinarily used in structures of this type, will not affect the final results materially.

J. R. Worcester, M. Am. Soc. C. E. (by letter).—This paper is a valuable contribution to the literature concerning building construction, and, undoubtedly, it will have the effect of improving present practice in a marked degree. At the present time, more steel is wasted in buildings than in any other branch of construction where steel is used. Bridge engineers have reduced designing to a science, so that the steel in a bridge is distributed where it is most needed, and there are few parts of a bridge structure that contain surplus material, but the engineering of steel buildings is in about the same condition of enlightenment that surrounded bridge building thirty years ago, when all parts of a bridge, from stringers to trusses, were designed for the same uniform load per running foot. Mr. Worcester.

Mr. Worcester. The reason for this is, largely, because engineers are not given a chance to use their discretion in the design of buildings, but are obliged to conform with building laws, which must be made so simple that they can be used by men without a technical training. Moreover, in the framing of the laws, it is necessary that they should be so simple that they can be understood by legislators having little interest in matters of construction.

The consideration of the subject by this Society, will, at least, have the effect of furnishing an authority for those interested in the revision of laws to refer to, and is particularly opportune at the present time, because, on account of recent conflagrations, the subject of building-law revision is prominent in many parts of the country.

The author, in dealing so exhaustively with his subject, has opened up a number of questions that should be carefully considered, and it is a question whether he has not gone a little too far in some of his radical recommendations of changes in present practice. The most important question suggested is in the reduction of the uniform load per square foot for floors. While the proposed single concentrations and loads per linear foot for girders will, in most cases, supersede the uniformly distributed load on floors, it does not seem to be wise to reduce this uniformly distributed load to a point below that which is likely to be placed upon any floor. The writer thinks that the author has laid too much stress upon the experiments by Messrs. Blackall and Everett, in Boston. These gentlemen investigated only three office buildings, and it is not likely that in taking three buildings at random, and making a single examination of each, really maximum conditions would be found. In determining the maximum loads on bridge floors, it would hardly be fair to select three bridges at random, measure up the loads found upon them at a certain time, and consider that the maximum of these three was all that would be necessary to provide for in designing new bridges. To be sure, the load, in the form of office furniture and fittings, might be said to be typical of that in all office buildings, but, in the writer's opinion, every part of every building should have capacity enough to carry the load of a crowd of people. Even in dwelling houses, all know that, occasionally, companies will assemble sufficient to make a fairly dense crowd in several rooms at the same time. According to the experiments* of Professor L. J. Johnson and Professor C. M. Spofford, it is within the bounds of possibility to get a load exceeding 150 lb. per sq. ft. from a crowd of people, while 80 lb. per sq. ft. represents not more than might be easily assembled at a social gathering. If any parts are designed for 40 lb. per sq. ft., with the usual unit stresses on the material, it is quite possible that the elastic limit might be reached without excessive crowding. To the writer, it seems as if 50 lb. per sq. ft., which

* *Engineering News*, April 14th and May 5th, 1904.

has been the loading required for dwellings, hotels, etc., in a number of cities, is low enough, though, with the alternative concentrated loadings, it seems as if this might be applied to office buildings and public assembly rooms, schools, etc. Mr. Worcester.

Taking up the detailed provisions of the specifications, the writer would suggest the following modifications:

Paragraph 6.—The words "are likely to," in the second line, should be changed to "can," and the word "distributed" should be inserted before "live loads," so that the paragraph would read:

"Flat roofs of office buildings, hotels, apartment-houses, etc., which can be loaded by crowds of people, shall be treated as floors, and the same distributed live loads shall be used as specified for hotels and dwelling-houses."

Paragraph 10.—The reduction of live loads on columns should not apply to warehouses which are likely to be loaded on all floors at the same time.

Paragraph 12.—It is an open question whether 30 lb. per sq. ft. is not an excessive wind pressure to allow on city buildings. Possibly, the author intends to allow a certain amount of discretion as to what should be considered "exposed surfaces." At any rate, it seems as if it would be safe not to require special wind bracing in steel frames of buildings of which the least horizontal dimension is as great or greater than the height.

Paragraph 13.—It would be better not to include "wet sand" with "soft clay," as wet sand frequently has much greater supporting power than 1 ton per sq. ft. The writer would recommend increasing the allowable load for ordinary clay to 3 tons, and for dry sand and dry clay to 4 tons, and for hard clay to 5 tons.

Paragraph 14.—The allowable pressure on hard-burned brick with Portland cement mortar might safely be increased to 15 tons. The pressures allowable for Portland cement concrete seem unnecessarily conservative; 30 tons per sq. ft. is safe with either grade of Portland cement concrete mentioned.

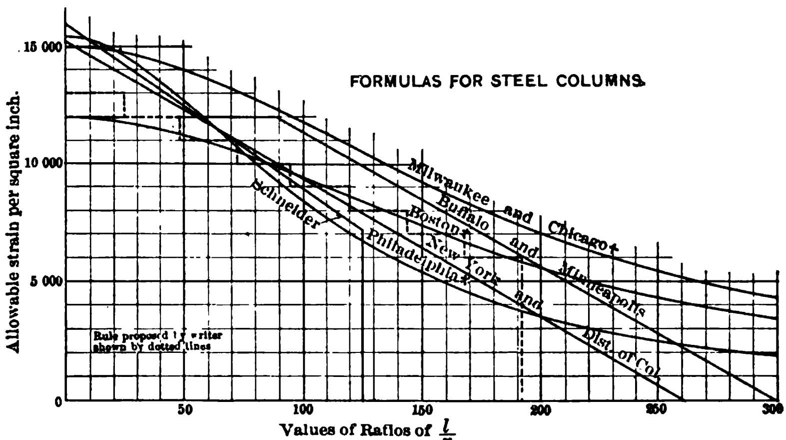
Paragraph 15.—The pressures under wall-plates on brickwork with cement mortar, or rubble masonry with cement mortar, might be increased to 200 lb., on Portland cement concrete to 400 lb., and on first-class sandstone to 400 lb. On granite, 800 lb. is all right, and it seems as if it were advisable to have one pressure specified instead of upper and lower limits.

Paragraph 19.—In adopting the bridge formula for compressive strains in columns, the author proposes very different units from those commonly allowed. The diagram, Fig. 3, shows the strain per square inch allowed by various formulas. The ordinates give the unit strain and the abscissas proportions l to r, between 0 and 300. It is evident from this diagram that there is a wide divergence of opinion as to the proper allowable units, and it seems to the writer that, where such

Mr. Worcester. great variations are found between different authorities, it is absurd to split hairs by calculating different unit strains for each variation of the value of l to r. As an alternative rule for compression, which would give in every case safe results and be much easier of application, the following is suggested:

"Columns may be used with a ratio of l to r not exceeding 16, l being expressed in feet and r in inches. The unit strain allowed shall be 13 000 lb. per sq. in. for $\frac{l}{r}$ from 2 to 4, 12 000 lb. from 4 to 6, with a decrease of 1 000 lb. for each succeeding increase of 2 in the ratio of l to r."

The result of this rule is also plotted in Fig. 3, and is shown by the stepped line. In the use of this rule, practically all the columns



* Average between values for medium and soft steel.
\+ Using Rankine's reduction formula.

FIG. 3.

in the same tier of a building would be calculated for the same unit strain, and this would be a round number, whereas a close figuring by any other rule would require a large majority of the columns to be calculated separately. The economy of this is readily appreciated by those who have had experience in designing building framework.

Paragraph 28.—The author's limit for compression members, 125 times the least radius, would in many cases necessitate a large increase of material, particularly in roof trusses.

Paragraph 29.—The rule of allowing the web to be calculated in the flange area of plate girders is unsafe without the provision that, where thus used, splices must be made at points where the total flange section is not required, or special provision must be made in splicing to transmit bending moments.

Paragraph 33.—A limit of one-fifteenth of the depth would in some cases infringe on architectural clearance in a way that would be very troublesome. The rule should be arranged so as to provide for beams of shallower proportions than this, by using properly increased unit strains. Mr. Worcester.

Paragraph 35.—As cast iron is allowable for column bases, even if nowhere else, a limiting tensile fiber strain should be specified.

Paragraph 37.—It would be interesting to know why the author considers it necessary to use, for wooden columns, a formula based on Gordon's, while, in the case of steel columns, he applies a straight-line formula. There is even more variety in requirements for wooden columns than there is for steel, as shown by Fig. 4, in which are

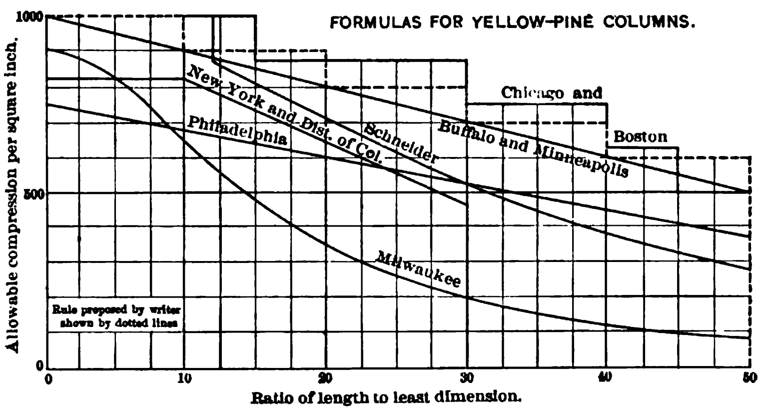


FIG. 4.

plotted the strains allowed on yellow pine by the cities quoted in the Appendix. The writer would suggest as a substitute the following:

> "Columns may be used with a length not exceeding 50 times the least dimension. The unit strain for lengths up to 10 times the least dimension shall be as given in Table 3, with a reduction of 100 lb. per sq. in. for every increase of 10 in the ratio of length to least dimension."

Paragraph 39.—It would be in the interests of permanence of construction if the limiting thickness of the material in the outside walls were made $\frac{5}{16}$ in., and where the protection is likely to be not more than 4 in., or, at most, 8 in., of brick, it seems as if this requirement should be insisted upon.

Paragraph 45.—A rule for the placing of tie-rods, based upon nothing except the depth of the beams, is extremely rough. In many kinds of construction where flat slabs are used, for instance, where the slab rests on top of the beam, tie-rods are practically superfluous;

Mr. Worcester. while, on the other hand, where segmental brick arches are used for floor construction, it is quite possible that the spacing of eight times the depth of the beam would be altogether too great for small rods. To make this paragraph correspond with the thoroughness of the remainder of the specification, it seems as if a rule should be given by which the tie-rods could be calculated properly. The only uncertainty on this point is as to how much dependence should be placed upon the support afforded by the arches in adjoining bays. It is unnecessary to calculate on the rods in one bay resisting the entire thrust from the arch of that bay, provided the bay is one of a series. As a simple rule, which is approximately correct, it is suggested that rods sufficient to resist half the total thrust should be provided.

Paragraph 49.—"Connection angles" may mean a variety of styles of connections, some of which are not desirable.

Paragraph 54.—The question might be asked: What should be provided in the way of splices if there is no bending strain?

Paragraph 57.—In many instances trussed purlins are economical and good practice. The reason for prohibiting them should be further explained.

Paragraph 61.—In members composed of two angles only, the maximum pitch, 6 in., is altogether too short, and does more harm than good.

Paragraph 80.—The application of this to building construction would be so rare as to make the clause superfluous.

Mr. O'Brien. JOSEPH H. O'BRIEN, ASSOC. M. AM. SOC. C. E.—This paper has proved so interesting that the speaker has taken occasion to review it quite carefully.

In what follows it is not the speaker's purpose to criticize the author, but merely to call attention to some points which seem to need more explanation than has been accorded them.

Mr. Schneider advises that all floor beams for floors of office buildings be tested for the maximum effect thereon of a concentrated load of 5 000 lb. (equivalent to the weight of the larger portable office safes). It is evident that if a safe weighing 5 000 lb. has any appreciable area, one floor beam (under usual conditions) would not support the entire weight; therefore, 5 000 lb. will be a liberal and not an actual value, if it is assumed to be concentrated on one floor beam.

Similarly, in regard to the load of 326 lb. per lin. ft. determined from the weight of a filled plan case, the author assumes the entire load on one floor beam, although he gives a width of 31 in. for his plan case. Of course, if someone should pile the cases two high, the author's assumed loading would be increased. However, his assumption of the entire 326 lb. per lin. ft. on one beam is so liberal that, doubtless, it would cover the possible condition just referred to.

The speaker's contention is regard to distribution of load is admitted by the author in the case of the book cases, but not in the case of the 5 000-lb. concentrated load. Mr. O'Brien.

The author states in the last paragraph of the section entitled "Wind Loads":

"In proportioning the members of the structure for these temporary wind strains, it is permissible to allow a higher unit strain than for permanent work, say 20 000 lb. per sq. in., or about two-thirds of the elastic limit."

The speaker has been accustomed to the use of some such liberal unit strain for wind bracing only, using the usual value for permanently loaded members, and he thinks this practice to be in conformity with a strict interpretation of the New York Building Code. The author would permit an increased unit value for strains due to wind on the entire structure. To carry out the author's suggestion it is only necessary to reduce the wind strains on permanently loaded parts of the structure in the ratio of the larger to the smaller unit value. For example, if the permissible unit strain for dead loads is 15 000 lb. per sq. in., and the supporting member considered sustains a wind strain at intervals equal to 60 000 lb., then fifteen-twentieths or three-fourths of 60 000 lb., namely, 45 000 lb., could be used as the wind strain to be added to the dead-load strains on the member, giving a total strain which, when divided by the working unit for dead load (15 000 lb.) would satisfy the author's recommendation.

If the author's suggestion in this particular be generally adopted it will be necessary to make the building laws more explicit.

In Paragraph 17 of the specifications, the author gives an item,

"Shear, on plate-girder web (gross section) 10 000 lb. per sq. in."

Before coming to New York City the speaker had been accustomed to use the gross section in determining the girder webs for building work, but when engaged in writing designers' specifications for some New York City work, the design of which he is directing, he made careful inquiry in regard to New York City practice in this particular, and found that, even those concerns accustomed to exceptionally economical frames used the net section in determining plate-girder webs. To be sure, the reduced strain in the gross section of the web, if web rivet holes are deducted, makes the girder proof against buckling in most cases and, except where good practice requires stiffeners for other reasons, they could be omitted.

It is so seldom, however, that increased web-plate section *versus* stiffeners comes up for consideration, as related solely to the plate-buckling value, that the practice of deducting web rivets seems to the speaker to be an outgrowth of some other consideration.

In searching for this other consideration, the speaker was confronted with the possibility of loose rivets in the end-bearing stiffeners, but, as the reaction gets into the girder through the medium of the rivets in the end connection or bearing, and the vertical forces in the

Mr. O'Brien. girder reach the abutment through the same medium, the loose rivets would cause an increased strain in the tight ones, and as these rivets if capable of resisting failure from such a strain would, if the plate tended to shear, meet the resistance of two vertical shearing surfaces in the plate, it appears that, for consideration of purely vertical shear, particularly with machine rivets (which are seldom loose) in the end stiffeners or connection angles, it is not necessary to deduct rivet holes in determining the web section.

Another point is that, possibly, in the case of a girder riveted between two columns, deflection would cause a tension in the web coincident with vertical shear, resulting in a tearing action which might cause failure at the end-riveted connection.

The speaker is convinced, however, that it is unnecessary, except perhaps for heavy railroad bridges (where very great rigidity is required), to use the net section in designing plate-girder webs.

Mr. Seaman. Henry B. Seaman, M. Am. Soc. C. E.—This paper is so thorough in its preparation that there is little room for criticism, but there are a few points of difference in practice to which attention might be called.

The provision of the New York Building Law (Paragraph 10), that the loads on the columns of buildings more than five stories high, be reduced, by considering that the lower floors carry the lightest loads, seems to give an erroneous interpretation to the original purpose of the modification from the full load. It provides that floors nearer the roof may be considered fully loaded, while those nearer the street would be considered only half loaded. This is contrary to probable conditions. The floors nearer the street would naturally be loaded first, and, in a crowded building, would be more likely to remain fully loaded than the floors above.

The purpose of the law would seem to be to provide for the reduction of the average simultaneous load of all floors, rather than merely the unequal loading of the different floors. The speaker, therefore, would suggest the following revision of the wording of Paragraph 10.

"For buildings more than five stories in height, these live loads may be reduced as follows:

"For columns supporting the roof and top floor, no reduction.

"For columns supporting each succeeding floor, a reduction of 5% of the total live load may be made, until 50% is reached, which reduced load shall be used for the columns supporting all remaining floors."

This will give slightly decreased sections, but it will make ample provision for safety, and, very evidently, was the original purpose of these laws.

The provision for uniform dead load pressures on the foundations is an excellent one.

The use of the empirical right-line column formula, instead of the rational one of Rankine, is unfortunate, although this form has come into quite common use. The empirical is not more accurate than the rational form, and is of more limited application, as shown by Paragraph 28. The two forms are usually reduced to tables, or diagrams, and are equally convenient. The step from the rational to the empirical seems to be a step in the wrong direction. Mr. Seaman.

The inconsistency of its adoption here is shown in Paragraph 37, where the Rankine form is used for timber.

The provision (Paragraph 25) that connections subject to alternate strains "shall be proportioned for the strain giving the largest section," does not seem to be sufficient. Reversal of strain is the greatest cause of deterioration in old structures. As soon as the strain in a member is relieved, or reversed, it gives opportunity for wear, and should require greater section than if the strain acted continuously in one direction and if it were constantly applied.

It is quite a common practice to allow one-sixth of the gross section of the web of plate girders to be available as flange area. The plate girder has an advantage in strength and stiffness over other forms of built beams, and, to restrict the use of web area to one-eighth (Paragraph 29) seems to be an unnecessary refinement, in this class of work.

The provision (Paragraph 49) that floor girders shall be riveted to columns by means of connection angles, implies that shelf-angle supports, alone, are not sufficient. It would be well, also, to avoid the use of bolts wherever practicable. Shelf-angle and bolt construction, now in quite common use, has little advantage over cast-iron column construction, at present so generally, though perhaps unjustly, condemned. Under these loose conditions, the wrought-iron column has not even the advantage of cast iron as protection in case of fire. It might be better, however, to specify that floor girders shall be preferably riveted to columns by web connections, since the connection to a Z-bar column, by making a splice connection of the diaphragm through the center of the column, to the web plate of the girder, makes an exceedingly rigid connection, especially desirable where the ironwork is expected to resist vibrations.

Under "Material and Workmanship," Paragraph 12 provides for modification in elongation, of material less than $\frac{5}{16}$ in., and more than $\frac{3}{4}$ in. in thickness. This might well be omitted. Material more than $\frac{3}{4}$ in. thick should not be used, and it is not worth while to make special provision for material less than $\frac{5}{16}$ in. thick. Thin material is generally used for fillers, and need not be tested. Concessions in required elongation of thick material usually mean that it receives less work in manufacture, and should receive less strain in design. It is a convenience in manufacture, but its use should be avoided. The special case of pins and rollers is provided for elsewhere.

Mr. Seaman. The use of pneumatic hammers, instead of hand riveting, insures much better work than formerly. Provision should also be made for the use of oil rivet-heaters, instead of the hand forge. When these were first introduced upon the New York Subway, several years ago, they both met with considerable opposition; but they won their way into favor, as being both economical and efficient, and have since come into very general use in the field. The labor unions object to the use of the oil rivet-heater, because one man may supply several gangs, but these heaters are so superior to the hand forge that their use is justified, if for only one gang

The provision that, in cutting out rivets, "great" care shall be taken not to injure adjoining metal, is a dangerous clause to place in the hands of an inexperienced or impractical engineer. If a well-heated rivet is used to replace the one removed, ordinary care would be sufficient, as the metal is not likely to be injured seriously.

The last eleven words of Paragraph 65, "unless he is not responsible for the design of the work," should be omitted, as a faulty design is always the designer's responsibility, and should receive protest before testing.

In closing, the speaker wishes to offer his thanks and his congratulations for a very well-considered and exhaustive paper.

Mr. Smith. AUGUSTUS SMITH, M. AM. SOC. C. E.—Mr. Schneider's opportune paper on the structural design of buildings discusses the present practice so concisely, and, in the comparative tables compiled from the building ordinances of various cities, discloses such a surprising diversity of opinion as to loads and allowable stresses, that it will undoubtedly do much to lead discussion on matters where opinions seem to differ so widely, and to crystallize the thought of constructors in this line of work, so as to produce a more uniform practice.

It is perhaps logical that the building laws of different cities should prescribe different loads within certain limits, depending upon local conditions, but surely the safe stresses for materials of construction need not vary for different localities.

The author very pertinently calls attention to the rational design of floors by considering the effect of a concentrated load, as well as a uniformly distributed load, and proportioning the floor joists and girders to meet the most severe condition. This method of designing is highly desirable, for all the reasons pointed out by the author, and for the further reason, which he did not mention, that it would be a more complicated method than the one now in vogue, under which every architect's office boy thinks he is competent to pick out the right-sized beams from a rolling-mill handbook, and thus might lead to the more general employment of competent engineers to design the framework of steel buildings, where mathematical computations are desirable.

Another point which the author has brought out very forcibly is the theory of designing foundations on compressible soils, with the area of the foundation made proportional to the dead load, considering the live load only as a factor in determining the allowable unit pressure on the soil. This is very scientific, and should lead to good results. Mr. Smith.

The emphasis laid by the author on the importance of considering wind pressure should do much to offset the carelessness in this respect that is manifested by many designers of buildings of moderate height.

Surely the framers of the present Building Laws of the City of New York overlooked such structures as pier sheds and freight sheds when they drew up Section 140, which ends as follows:

"In buildings under 100 ft. in height, provided the height does not exceed four times the average width of the base, the wind pressure may be disregarded."

At the time these Building Laws were being discussed, the speaker attended a "public hearing," arranged by the Board of Aldermen for the purpose of entertaining any objections that might be made to the proposed Law, and argued quite at length on the defect in this section on wind pressure. Unfortunately, a delegate from the Plumbers' Union was next heard and made such a fuss about certain provisions of the law requiring less assistance from the plumbers than might have been called for that the defect pointed out by the speaker was entirely forgotten.

It is fortunate that Mr. Schneider has brought up the question of wind pressure in the way he has. As stated at the beginning of his paper, he invites discussion and criticism of the proposed specifications and of the many new statements and figures contained therein.

Confining this discussion to "Part I—Design," the speaker has the following suggestions and criticisms to offer on the subject matter of the following paragraphs:

Paragraph 3.—Table 2 proposes a new requirement, for loads in buildings, which is intended to be more nearly in accordance with the actual loads which the buildings will be called upon to sustain. In office buildings, a proposed uniformly distributed load of 40 lb. per sq. ft. of floor is called for. The average of the requirements of eight cities, in this respect, as given in Table 5, is 82 lb. per sq. ft.

The application of Table 2 will show, as pointed out by the author, that in most cases the floor would be designed for the concentrated loads and loads per linear foot on girders, whether the distributed load were taken at 40 or 80 lb. per sq. ft. of floor, and the concentrated loads and loads per linear foot seem to have been chosen very wisely. Therefore, the following remarks about the distributed load are somewhat academic. Nevertheless, the light distributed loads prescribed for office buildings, assembly rooms with fixed seats, and

Mr. Smith. ordinary stores, at first glance look like a radical departure from present practice, and seem to the speaker to have a bad "moral effect" For instance, the architect's office boy may still venture on the problem and get his figures tangled up so that a floor may be designed for 40 lb. per sq. ft. instead of for the concentrated loads prescribed by the table, and it is easier and safer to avoid such a mistake, by prescribing a heavier distributed load, than it would be to detect the mistake after the plans were out.

The real reform aimed at by revising the distributed loads in Table 2 is probably in the design of columns, as would appear later by the provisions of Paragraph 9. This reform, however, could doubtless be accomplished in some other way.

A distinction is made in Table 2 for distributed load between assembly rooms having fixed seats and assembly rooms without fixed seats. To the speaker it does not seem proper to make this distinction, because a dancing floor is frequently constructed over the tops of the seats in such rooms, and, even when the seats are used, an audience is quite apt to stamp in unison, which would have nearly the same effect on the floor as dancing or marching would have.

Again, ordinary stores or light manufacturing buildings would seem to involve very similar loading to office buildings, and 40 lb. for this service seems to be very light. The average of the requirements of nine cities given in Table 5 is 119 lb.

The speaker agrees with the author that, in many cases, the loads assumed for floors have been excessive, and thinks that all the author has said about reducing the load on columns and on foundations is sound, but when there is so much money available for ornamentation in office buildings, there does not seem to be sufficient reason for lightening the floor construction in buildings of this class as much as Table 2 might allow in certain cases.

In dwellings, on the other hand, there is a positive advantage in economizing the design so that steel-framed buildings may be within the scope of the usual appropriation. The only reason why steel-framed dwellings are not more generally adopted is the higher cost, and the loading prescribed in Table 2 will do something to reduce this and thus encourage the construction of dwellings of this class.

The speaker would suggest, therefore, that office buildings, assembly rooms (with or without fixed seats), and ordinary stores and light manufacturing buildings, should be grouped together and designed for a distributed floor load of not less than 75 or 80 lb. per sq. ft., with the further provision for concentrated loads and girder loads given in Table 2.

Paragraphs 7 and 8.—There would seem to be a typographical error in placing the weight of corrugated sheeting on purlins at 50 lb. and gravel on boards having a steep pitch at 40 lb. Why should not the dead load of roofs be calculated as provided for floors in Paragraphs 1 and 2, and only the live loads discussed in Paragraphs 7 and 8? Mr. Smith.

For the past eight years, the speaker, with unvarying satisfaction to himself and his clients, has used the general specifications for roofs and iron buildings, by Charles Evan Fowler, M. Am. Soc. C. E., of which a fourth revised edition was issued in 1901. The loadings of Fowler's specifications, both for wind and snow, seem to be preferable to the loadings given in Paragraphs 7 and 8.

Paragraph 9.—The distributed loads prescribed in Table 2 were doubtless selected with the design of the columns in view, and, if changed as suggested by the speaker, would involve the rewording of this paragraph.

Paragraph 12.—A wind pressure of 30 lb. per sq. ft. is called for in the New York Building Laws, for buildings more than 100 ft. high, with an allowable unit stress of 50% more than for dead or live loads. Fowler gives 20 lb. for buildings less than 20 ft. high and 30 lb. for buildings 60 ft. high, with no extra allowable unit stress.

The author prescribes a wind pressure of 30 lb. per sq. ft., with an excess of 25% in the allowable unit stresses. and provides further that the framework shall be figured as an independent structure and designed to resist wind without walls, partitions or floors. It is obviously true that the wind pressure cannot be quite as great near the ground as it is some distance above it, and it is also probably true that large areas cannot be subjected to the same pressure per square foot as a small area would be. For instance, the flat side of a building must have a wedge of dead air in front of it, which would probably have the effect of reducing the total wind pressure on the projected area perhaps one-half. On the other hand, the small areas of the frame of a structure do not have this advantage, and, on these small areas, would it not be better to take 50 lb. per sq. ft., as it is commonly specified for wind pressure on bridges? It would appear, also, to make simpler computations if the assumed wind pressure on the side of the building were reduced and no additional unit stress allowed in the frame, which is the method of computation adopted by Mr. Fowler.

Paragraph 13.—The permissible pressure on foundations is so difficult to describe in general specifications and, in a certain sense, being not quite relevant to the design of the building itself, that it might be advisable to omit Paragraph 13 altogether. If, however, some specification for pressure is desirable, the depth of the foundation should enter as a factor. To borrow some words from Naval Architecture, it is apparent that the buoyancy of the soil is a function of the depth of the displacement of the building or other structure.

Paragraph 14.—It is to be noted that the author has given Port-

Mr. Smith. land cement concrete considerably higher value than bricks or rubble masonry laid in Portland cement mortar. If the allowable values given by the Building Laws of the City of New York are correct, the proposed value of hard-burned brick and rubble in Portland cement mortar, as given by the author, would seem to be low.

Paragraph 16.—No formula has been suggested for determining the bearing power of piles driven by a steam hammer. The speaker is not prepared to suggest any formula for this class of work, but would call attention to the desirability of one.

Paragraph 21.—There would seem to be a typographical error in prescribing 600 d for the pressure on expansion rollers per linear foot. Probably per linear inch was intended. The late J. B. Johnson, M. Am. Soc. C. E., gives for this pressure $P = 1\,200 \sqrt{d}$. If d were taken as 1 in., the author's formula would give one-half of Johnson's allowable load, but if d were taken equal to 4 in., the allowable pressure would be the same by both formulas. From a superficial inspection, it would seem to be more logical to take the pressure directly proportional to the diameter than to the square root of the diameter.

Paragraph 22.—It seems to the speaker unsound to treat a combination of transverse loading with tension or compression by the same formula. Certainly, transverse loading of a compression member is more dangerous to the safety of a structure than the transverse loading of a tension member.

Paragraph 29.—The speaker would prefer to have the second sentence read:

"The compression flange shall have at least the gross sectional area of the tension flange, etc."

Paragraph 30.—The speaker would suggest adding the words, "and shall preferably be designed as a strut with transverse loading with adequate moment of inertia laterally." The speaker happens at the present time to be strengthening a crane run-way in which the length of the compression flange was 22 times its width, but in which, if the flange had been one-twentieth of the length of the girder, the result would probably not have been satisfactory.

Paragraph 33.—The limiting depth of roof purlins, if continuous over more than one span, may be less than one-thirtieth of the span and give satisfactory results. The speaker's practice for continuous purlins over three supports, has been:

4-in.	purlins	for	12-ft.	span,	
5 "	"	"	14 "	"	
6 "	"	"	16 "	"	
7 "	"	"	18 "	"	and
8 "	"	"	20 "	"	

The last size of purlin is one-thirtieth of the span, but the others are less.

Paragraph 42.—The speaker would suggest making this paragraph read: Mr. Smith.

"The strength of connections shall be sufficient to develop the full strength of the member, except in the case of shapes used in tension, or where extra material has been used to reduce deflection or distortion."

Paragraph 45.—The speaker would call attention to the fact that $\frac{1}{80}$ in. per foot of span for a fire-proof floor is a very considerable deflection to allow if the load producing it is apt to occur frequently.

Paragraph 49.—It would be wise to add to this paragraph:

"If shelf angles or other supports are calculated to take any share of the load imposed by the end of a beam or plate girder, the beam or girder shall be riveted securely to the shelf angle or other support."

The object of this requirement is to assure the bearing on the shelf angle that was assumed in the calculations.

Paragraph 56.—The provisions of this paragraph are frequently difficult to meet, in the case of the intersection of the top and bottom chords of flat triangular roof trusses, without making the depth of the truss over the supporting column so shallow as not to provide sufficient metal to take care of the shear. It would be desirable to add, to take care of this and other similar cases, the words:

"And when this is impracticable, the resulting eccentricity must be computed and provided for."

Paragraph 57.—In the case of long roofs, such as pier sheds, it is not necessary that all the roof trusses shall be braced in pairs, for the reason that the purlins or roof planking can be relied upon to sustain intermediate trusses, if, at suitable intervals, a pair of roof trusses be braced together. Perhaps it would be sufficient to obtain good construction if this paragraph were made to read:

"At least 50% of the roof trusses of a building shall be braced together in pairs at suitable intervals in the plane of the top chord, if the roof covering is carried on purlins or jack-rafters."

If roof planking be secured directly to the trusses, which is frequently the case, the planking can be relied upon for sufficient bracing.

There would seem to be no gain in bracing the bottom chords of the roof trusses, unless in special cases it should be necessary to take care of the wind pressure against the sides or ends of the building by bracing in the plane of the bottom chord. Roof trusses of the old style, with round wrought-iron rods for tension members, were not braced in the plane of the bottom chord, and they seem to get along quite as well as trusses of the new style, built of angle iron, where the convenience of the connection seems to tempt the designer to add a lot of bracing.

The bottom chord of a roof truss is in tension, and, necessarily, takes the shortest line between the points of support, and will stay

Mr. Smith. there, in consequence of the load on the truss, without any bracing. The requirements of Paragraph 57 would seem to call for unnecessary bracing.

Paragraph 79.—Expansion rollers, considerably less than 4 in. in diameter, would seem to have a legitimate field for use under such girders and trusses as are apt to be found in building design.

Paragraph 81.—The wording of this paragraph would indicate that columns not strained in tension at their base need not be anchored to the foundations. It has always seemed to the speaker advisable to anchor, or at least dowel, columns to foundations, whether or not they are strained in tension, for the reason that changes of temperature or minute vibrations, by moving the base of the column, always in the direction of the least resistance, would tend to make it "creep" off the foundation, and, although this would be impossible in the case of a large building supported by many columns and sunk some distance in the ground, it would possibly tend to set up transverse stresses in the unanchored columns to keep them in place on their foundations.

Mr. Coombs. R. D. Coombs, Jr., Assoc. M. Am. Soc. C. E. (by letter).—The writer does not think that the 40 lb. per sq. ft. uniform live load, for hotels, theaters, schools, etc., in the author's very timely specifications, is high enough. At least, it would seem that a greater value should be used in hallways, lobbies and assembly rooms, of theaters and college buildings. If the calculated load, 156 lb. per sq. in., obtained by Professor Spofford, did not require uncomfortable crowding, then much higher loads than 40 lb. must frequently occur.

Students leaving a classroom, coincident with the arrival of the coming class, would increase the load carried by the hallway beams and girders to more than this figure. In class "rushes," whether impromptu or premeditated, coming under the writer's observation while in college, the men were "very uncomfortably" crowded over a hall area of about 300 sq. ft.

With the possibility of similar loading, it would seem desirable to specify a live load of 100 lb. per sq. ft. for beams and girders.

Paragraph 65 of Part II.—This paragraph, as worded, would relieve the contractor of the cost of members which might be faulty in design and deficient in unit strength; whereas the latter consideration should cause the rejection of the material. As a modification, it is suggested that the paragraph be worded:

"If it does not develop the specified unit stress at the point of maximum stress, it will be considered rejected material and be solely at the cost of the contractor."

Mr. Llewellyn. F. T. Llewellyn, Assoc. M. Am. Soc. C. E.—This paper forms a valuable contribution to engineering literature, particularly because it deals with a branch which is of the utmost importance to the capitalist, the engineer, and the occupants of buildings, but which, strangely enough, has received much less critical investigation and is based on a more variable practice to-day than almost any other line, unless it be that of reinforced concrete. In both these subjects engineers seem to have reverted to those old Hebrew times when every man did that which was right in his own eyes, and naturally wrong in his neighbor's. The ingenious methods proposed by Mr. Schneider must tend to bring every man and his neighbor into greater uniformity, and along lines that seem self-evidently reasonable. There may be difference of opinion regarding the precise figures to apply as floor loads, working stresses, shop practice, or quality of material, but the method of proportioning the structure, from the floor joists down to the foundations, is so logical, and in line with what has proved proper in bridge construction, that it promises to result in greater conformity and security, and with better disposition of material, at probably no greater cost. It is to be regretted that these specifications could not receive the test of actual use as a basis for more intelligent criticism at this time. Mr. Llewellyn.

The particular feature of the specifications which the speaker proposes to discuss is their practical result, which appeals to those specializing on structural work. The floor joists would be generally somewhat heavier than in existing practice; short beams would be considerably heavier, and floor girders would be somewhat lighter. These modifications would reduce the total thickness of floors, and minimize projecting girders, which, in hotels and loft buildings, where partition arrangements are subject to change, is very desirable.

Unduly light coping and other connections would be eliminated, resulting, not only in greater strength under concentrated floor loads, but also allowing more security to the erector in the way of derrick fastenings and guy-line hitches, for the man on the job will hitch to the handiest piece, which is also generally the smallest. Increased uniformity in coping connections would be feasible, permitting the use of multiple punches, and facilitating the preparation of shop details. There would be less variety in the sizes of beams required, thereby securing quicker rollings of material at the mills. There would be less violent changes in the column sections from story to story, to the lasting happiness of the shop draftsman, and also permitting fewer sizes of fire-proofing and masonry. The proposed method of proportioning foundations seems to be admirable, especially where rock is not reached.

Paragraph 16.—The explanation, " $W =$ weight of hammer, in tons," should be inserted in this paragraph.

There is an apparent discrepancy between the allowed use of Bessemer steel in Paragraph 2 of Part II and the chemical requirements of Table 4.

Mr. Llewellyn. In Paragraph 1 of Part II cast iron is allowed for column bases and bearing plates, and in Paragraph 35 of Part I its allowable unit stress in compression is given; but, in proportioning the thickness of such plates, bending must be calculated, and the allowable unit stress in tension should be inserted. It is suggested that 3 000 lb. per sq. in. would be proper, or a combined bending stress of 5 000 lb. per sq. in., on the average cross-section between ribs, would allow ready calculation, with fair, if not accurate, results.

The most serious omission, however, occurs in Paragraph 21 of Part II. The specifications allow the use of cast iron for base plates, but do not stipulate closely enough the necessary safeguards, and that, too, in items which sustain the entire weight of the building. Provision is made for tests and finish, but there is silence regarding the most important feature of cast iron, namely, its inspection at every stage, and particularly an early morning inspection of castings, poured over-night, while being shaken out. One great advantage of rolled steel is its accessibility to inspection at every stage, and cast iron, if used at all, should be similarly accessible. To illustrate the need of this inspection, one need but glance at the advertising pages of any foundry journal, where such names as "Smooth-it-over," "Pile-it-on," or "Filler-up" are given to substances sold for the purpose of concealing defects in castings. The foundry-man welcomes visitors to inspect the mysterious methods of sand or sweep moulding, or to witness the sprays of sparks that fly from the molten metal while a heat is being taken off at night, but one should see the result in the "cold gray dawn of the morning after," before it can be doctored.

During the middle ages progress was slow, for the reason that all crafts were secret. The advance in the modern use of steel is due largely to the opposite policy, and the facilities for inspecting the product at all stages, not only ensure its reliability, but offer opportunities for each steel master to improve on his neighbor's methods; nor have they been slow to do so. One reason why there is to-day so much doubt as to the safety of cast iron (an excellent material in its proper place) is because short-sighted iron-founders have attempted to force its use where it has no place, and have bolstered up their claims by statements which cannot be investigated on account of their secretive policy in reference to its most important period, namely, just after cooling. Until this attitude, a relic of medieval barbarism, is removed, it will not be possible to estimate properly the real value of cast iron as an engineering material.

Some of the very valuable tables placed in the appendix to this paper should be removed to the body of the specifications for clearer reference.

Mr. Cooper. THEODORE COOPER, M. AM. SOC. C. E. (by letter).—It is very desirable that specifications for the structural features of buildings should be brought into some general uniformity based on technical common sense. Of course, absolute uniformity is not to be expected, for even experts are not yet in harmony as to the minor details of any class of construction. Mr. Schneider has done a good work in bringing so important a subject up for consideration and discussion. Mr. Cooper.

If the discussion tends to produce any substantial agreement as to live loads to be used for the various cases of everyday practice, as it should, a great advance will have been made in building practice. On the subjects, unit strains, details of construction, shop practice and material specifications, there are less differences, differences which it would be useless to discuss until there is a reasonable agreement as to the live loads which are to be used.

In practice, it is the concentrated loads which determine the strength or safety of the floors and building. Uniform loads, sufficiently high to cover the concentrated loads, produce wasteful construction, without any compensating benefit to the building.

The method proposed by Mr. Schneider, adopting a uniform load sufficient to cover all cases, where the load may be uniformly distributed, and then supplementing this with a concentrated load to provide for any excessive local loadings, is in the line of economic and therefore good engineering.

The distributed load of 40 lb. adopted by the author for people and ordinary fittings in rooms and offices is, in the writer's opinion, a liberal allowance. Rooms are not loaded by dropping the last possible person into the seething mass below by means of tackle, as has been done, to determine the weight of crowds.

In an assembly room of any kind, great local concentration of people may be caused by a fire, fight, or panic, yet the load over the whole floor will not be increased. Most people have experienced the discomfort of a crowded Elevated Railroad car, when not another person could be squeezed inside of the gates. Such a crowd numbering about 120 persons and not weighing more than 18 000 lb. is contained in a space of about 400 sq. ft., including platforms, or 45 lb. per sq. ft.

A weight of this kind would not be expected in living or office rooms, theaters, churches, schools, armories, ballrooms, etc., over the whole floor. A popular reception or a panic might produce this, or a somewhat larger loading in the aisles or corridors, or on the stairways, but this would be taken care of by the concentrated load.

The author has allowed for 100% impact and vibration, or has increased his uniform local to 80 lb. for ballrooms, drill rooms, etc. This is liberal, for when people are packed in so as to weigh 40 lb. per sq. ft. of floor, there will not be much marching or dancing. Instead of this allowance of 100% in all cases, it would be better and more just

Mr. Cooper. to make the allowance variable with the dead weight of the floor, as heavy solid floors should have an advantage over those of light weight.

The concentrated loads adopted by the author appear to have been well selected for the several cases.

Paragraph 13.—In this paragraph the word "permissible" should be omitted, and the words "Pressure on foundations not to exceed" used instead. It is not safe to define the permissible pressures on a foundation solely on a general classification of soils. Limiting pressures may be specified, subject to reduction by local experience or examination.

The classifying of soft clay and wet sand together must be a typographical error.

Paragraph 37.—The straight-line formulas for timber, as deduced from the Watertown tests on timber, should be used, as a simpler form and based on actual tests.

Mr. Post. HENRY W. POST, M. AM. SOC. C. E. (by letter).—It is to be hoped that the presentation of this paper will lead to the adoption, throughout the country, of a uniform set of standard specifications covering all systems of building construction.

In view of the extremely short time usually allowed an engineer for designing the structural portion of a building, any general information which can be embodied in the specifications, or any clause which will lessen as much as possible the amount of calculation involved, will contribute to save valuable time. In the matter of dead loads it seems as if, considering the large number of well-known systems of floor construction, they might be divided into groups or classes with an approximate dead-load value for each class, as, for example, flat-tile arches, segment arches, concrete slab construction, etc., to include in each case the weights of all the material to make the finished floor.

The weight of partitions often forms a very large proportion of the dead load, and, as frequently happens, their location is materially changed after the structure is completed. Under such circumstances it would seem advisable to provide in the calculations, not only for the partitions as they are shown on the plans, but also for every other possible location, or else make suitable provision in the assumption of the live load.

The live-load units, as specified in most of the building laws, seem to be excessive, but, in the writer's opinion, the dead-load units for floors are often guessed at, and the partition weights neglected altogether, so that the result given by the combined loads is not excessive.

The following live-load units are suggested:

For apartments, dormitories, dwellings, hospitals, hotels, etc., 40 lb. per sq. ft. or 2 000 lb. concentrated at any point.

For schools, theater galleries, and churches, 60 lb. per sq. ft. Mr. Post.

For office buildings, above the ground floor, 60 lb. per sq. ft., or 5 000 lb. concentrated at any point.

For ground-floor offices, stores, light manufacturing, stables and carriage-houses, 80 lb. per sq. ft., or 5 000 lb. concentrated.

For assembly rooms, main floors of theaters, armories, and their corridors, or for any room likely to be used for drilling or dancing, 100 lb. per sq. ft.

For sidewalks in front of stores or warehouses, it is not uncommon to see large quantities of merchandise piled up, or heavy machinery carried over, so that a load of 250 to 300 lb. per sq. ft., or a concentrated load of from 8 000 to 10 000 lb., does not seem excessive, but, for sidewalks in front of dwellings, a much lighter load might be specified.

For lofts, storage, printing houses, or for heavy manufacturing purposes the live load should be determined by the requirements of the business.

As to the bearing of beams or girders on walls, it is suggested that for convenience the area of the bearing be made to bear some relation to the size of the beam used. Referring to the tables of the strength of beams in the Mill Handbooks, and taking the maximum safe loads for the shortest spans given, the end reactions are such that, if the area of the template required is equal to the square of the depth of the beam, the pressure will not exceed 250 lb. per sq. in. (except possibly in the heaviest sections of 12 and 15-in. beams, which are rarely used). As the beams are usually built into a solid wall at comparatively long intervals it would seem that the pressure of 250 lb. per sq. in. would be well within the limit of safety.

A length of bearing on the template of two-thirds of the depth of the beam would be ample.

Would it not be well to embody in the specification one or more clauses relating to furnishing for record the data upon which calculations are based, with such diagrams or stress sheets as may be necessary? It frequently happens, where alterations are made to existing structures, that such information is necessary, and is rarely to be found.

GUNVALD AUS, M. AM. SOC. C. E. (by letter).—This paper has given Mr. Aus. the writer great pleasure, as it recognizes many of the objections to the common practice of designing in accordance with the empirical rules laid down by the different building codes.

The question of materials to be used and loads to be supported was discussed in the Brooklyn Chapter of the American Institute of Architects in February, 1904, and a paper, presented by the writer on that occasion, was published in *Engineering News* of April 14th, 1904. Examination of that paper will show the writer's opinion on

Mr. Aus. most of the questions discussed by Mr. Schneider, so that it is unnecessary to discuss them in detail now and give reasons for that opinion.

The writer thinks that all engineers should agree to the general propositions advanced in the paper above referred to, and Mr. Schneider's admirable specification, namely:

First.—That floor beams should be designed both for a uniform load and for a concentrated load, to prevent the use of very light beams of short spans.

Second.—That floor girders—that is to say, floor members—which carry a considerable floor area should be designed for smaller live load than that for which the floor beams are designed, both because the entire area of the floor carried by such girders will never be fully loaded, and also because the loading on such girders accumulates so slowly as to do away entirely with the effect of impact to which the individual beams will always be subject.

Third.—That the columns in the lower stories should be designed for a gradually decreasing live load, as it is not within reason that all the girders supported on these columns will receive the maximum loading at the same time.

Fourth.—That the foundations should be designed for only a part of the live load coming on the basement columns, as otherwise unequal settlements will occur.

Fifth.—That the framework of a skeleton building should be so designed that it can resist wind pressure.

Sixth.—That preferably no cast iron shall be used in columns or lintels, but that cast-iron columns in no case shall be used in buildings more than four or five stories in height, and, when so used, that the ratio between the length and diameter of the columns shall be very much smaller and the permissible unit stress also very much smaller than is now allowed, for instance by the New York Building Code.

It is not easy to state just how big should be the uniform load and the concentrated load for which the beams should be designed, and the writer thinks the opinions of many engineers of experience should be heard before establishing these loads.

Would it not be advisable for the American Society of Civil Engineers to appoint a Committee to examine this question and make a recommendation, which in all probability would have great influence when there will again be a chance to modify the present Building Codes?

The writer is of the opinion that the uniform loads specified in Mr. Schneider's specification are ample, but he believes that the concentrated load and the load per linear foot are too great.

Safes weighing 5 000 lb. are used so rarely in ordinary offices that it would seem unreasonable to design every part of a building strong enough to support such a large concentrated load, and it is undoubtedly true that safes weighing 2 000 lb. cannot be found in 1% of the residences erected. Therefore, it seems to be unreasonable to design all residences for this excessive loading at an enormously increased cost, the more so as such a provision will tend to retard the movement toward fire-proof dwelling-houses. Mr. Aus.

The writer believes that the 2 000 lb. concentration suggested by him for offices and the 1 200 lb. suggested for residences would be ample. Special permission should be obtained from the Building Department in those few cases where heavier concentrations are to be supported.

It also appears that Mr. Schneider's specification, if adopted by the Building Department, would induce the designer, for the sake of economy, to use long spans, which, unless the design was carried out by experienced engineers (which is not always done) would tend to weaken the building materially. The typical office building or apartment-house erected to-day has beams of from 15 to 16 ft. span, spaced about 5 ft. on centers, which, under the author's specification, would call for a uniform live load of 133 lb. per sq. ft.; whereas, if the spans were increased to 30 ft., the live load would only be 66 lb. per sq. ft. Further, if girders of 30 ft. span, spaced 15 ft. on centers, supported beams 5 ft. apart, these girders would only be designed for half the live load for which the beams are designed, which appears to be too great a reduction.

The writer has for many years, as Chief Engineer of the Supervising Architect's office, in Washington, D. C., reduced the live load on the girders to two-thirds of that for which the beams were designed, and this appears to be as far as one should go in this reduction.

There is no objection to any other features of the author's specification, except that the advantage of changing the specification for structural steel from that now commonly used is not quite evident; that is, from 60 000 to 70 000 lb. ultimate strength, to that specified by the author; that is, from 55 000 to 65 000 lb. Practically all the steel which the writer has had inspected for years past, under the former limits, has had an ultimate strength of from 60 000 to 65 000 lb., so that it is not thought that a different material would be obtained under the author's specification; and, even if the material should go as high as 70 000 lb., which is very rarely the case, it can be punched and sheared readily without affecting the strength of the finished members.

J. K. Freitag, Assoc. M. Am. Soc. C. E. (by letter).—In view of the Hotel Darlington disaster and similar loose methods of building design and construction, the author is to be complimented on his Mr. Freitag.

Mr. Freitag. timely presentation of this very important subject, and it is to be hoped that this paper may serve the purpose of leading to a general revision of the building laws of large cities, with especial reference to uniformity and modern practice. The same wide divergency in building regulations in the more prominent American cities has been pointed out and discussed at some length by the writer.*

As regards the floor loads suggested by Mr. Schneider, the writer heartily agrees with the proposed live load of 40 lb. per sq. ft., for ordinary cases. The small live loads which have been found by such experiments as those conducted by Mr. E. C. Shankland and by Messrs. Blackall and Everett, ranging from 12 to 16 or 17 lb. per sq. ft., have tempted in some cases the use of unit loads as low as 20 lb. per sq. ft., but such recommendations should certainly be questioned and even heartily condemned in conservative practice. While 20 lb. per sq. ft. may be sufficient for average present loads in office buildings, etc., it is to be remembered that the use of an average is always dangerous, while provision should be made properly, but not extravagantly, for all possibilities of excess, either present or future. The character of a building's contents or usage is subject to extreme change. The entire building, or possibly only portions thereof, may be devoted to very different uses from those primarily assumed, so that, in spite of the provisions in building ordinances against radical change in the character or degree of floor loads, it is often difficult to balance present economy against possible maximum requirements or future possibilities. Here, as well as in the strength of materials, a sufficient factor of safety should always be applied.

The author proposes to calculate all floor beams for a concentrated load of 5 000 lb. at any point, this being the approximate weight of the heaviest portable safe which would commonly be used in offices, etc. A safe of this weight would be approximately 4 ft. wide by 3 ft. deep. The factor of safety assumed in the calculations of the floor beams, as recommended in Mr. Schneider's specifications, would be about 4. Professor Rankine† gives a factor of safety of 4 which would be applicable to a reliable steel-frame structure, while for masonry a factor of safety of 8 is recommended under live loads. Again, on page 361, the same authority states as follows:

"The factor of safety in structures of stone should not be less than 8, in order to provide for variations in the strength of the material. as well as for other contingencies. In some structures which have stood it is less; but there can be no doubt but these err on the side of boldness."

All present types of fire-proof floor arches would come under the classification of the poorest kind of masonry, so that, if the floor beams were calculated for a concentrated load of 5 000 lb., consistency would

* "Architectural Engineering," edition of 1901.

† "Civil Engineering," page 222.

require an ultimate arch capacity of 40 000 lb. applied over any bearing area of 12 sq. ft., or 3 333 lb. per sq. ft. Many, if not most, forms of fire-proof floor arches now in general use would fail to develop this strength. In the Denver tests, the best end-construction arch sustained a final load of only 15 145 lb., or 1 670 lb. per sq. ft. over a loaded area of 9 sq. ft.; while the best side-construction arch carried only 8 574 lb., or 953 lb. per sq. ft. over a loaded area of 9 sq. ft. These were 10 in. terra-cotta arches of 5-ft. span, hence they were fair samples of modern use. The tests made by George Hill, M. Am. Soc. C. E.,* on Melan and terra-cotta arches, by means of a self-registering hydraulic machine, show only one terra-cotta arch out of eleven tests which would fulfil the necessary requirements, *viz.*, a 10-in. hard terra-cotta end-construction arch, which sustained a total load of 57 500 lb. over a loaded area of 20 sq. ft. out of a total arch area of 22.6 sq. ft. The other ten arches ranged from 10 000 to 16 000 lb., ultimate loads. All the Melan concrete arches showed an ultimate capacity of more than the required 40 000 lb. Most, if not all, of the New York Building Department tests would also have failed to show this ultimate capacity, which would be necessary, not only under normal conditions, but under fire and water tests as well. Several of the concrete arched forms now in common use will probably exceed this required strength, but if the floor construction as a whole were to be proportioned equivalent to the concentrated load assumed for the beams, this might preclude the use of practically all flat terra-cotta arches and all slab concrete construction. Mr. Freitag.

The enumeration of dead loads includes partitions, while Paragraph 2 of the specifications for Design state that the calculations of dead loads are to be based on the weights of different materials given in Table 16. It would seem as though this table could properly be extended to give the weights of terra-cotta and mackolite partition material for different thicknesses, as well as some general data regarding the weights of terra-cotta arch blocks of different materials and for different arch depths. Partitions are often classed as live loads, and in previous Chicago practice these were often assumed to be live loads distributed uniformly over the floor area at 25 lb. per sq. ft. This was on the assumption that the position of partitions might frequently be changed to suit the subdivision of office areas as required by tenants. The present New York and Chicago Building Laws, however, both require partitions to be considered as dead loads.

As pointed out by the author, the live loads on warehouse and factory floors will vary greatly, these often being far greater than might be expected, and often much less. Mr. W. L. B. Jenney, the well-known Chicago architect, had occasion to estimate the loads in the

*"Tests of Fire-proof Flooring Material," *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 542.

Mr. Freitag. wholesale warehouse of Marshall Field & Co., in Chicago, and the low average of 50 lb. per sq. ft. was found for the total floor area, including all passageways, in spite of the very large quantities of merchandise usually stored there. The maximum load on limited areas was found to be 57 lb.

In the proposed specifications, cast iron is "practically ruled out" as unreliable and unadaptable material. This elimination of cast-iron members would be warranted if all building laws which now permit its use could be changed so as either to eliminate cast-iron columns altogether or to require the calculations of columns by reliable formulas, in which latter case there would be found to be slight, if any, economy in using cast iron; but, as long as present building laws exist, allowing the use of cast-iron columns, the writer can see no serious objection to using them under approved methods of calculations, for buildings of medium height and considerable area where wind bracing is not required beyond the stability of enclosing and partition walls. There is now under process of construction a large department store in Boston, approximately 200 by 250 ft. in area, seven stories high, besides the basement and sub-basement, with masonry walls and cross-walls. In such a case, the writer can see no objection to the use of cast-iron columns, if proportioned and designed properly.

It would also seem that Paragraph 12, relating to wind pressure, should be modified so as to limit the necessity of caring for wind pressure to those structures which would require metallic bracing. Inasmuch as the specifications refer to the structural steelwork of buildings, it would appear as though Paragraph 12 were intended to require a wind pressure of 30 lb. per sq. ft. to be calculated for all buildings having either complete or partial steel frames. This provision would not be necessary for buildings provided with exterior walls of masonry and with masonry partitions or cross-walls, especially if the base is a large proportion of, or equal to, the height.

Paragraph 54, regarding column splices, might be extended to require the breaking joints of columns alternately at any floor level.

Paragraph 57 of Part 2, regarding field painting, should be amended to require the field painting to be of a different color from the shop coat.

The intimate relation which exists between the steel frame of a modern building and the fire-proof floors and coverings makes it indispensable that the successful designer of the steelwork be also thoroughly familiar with approved fire-proofing methods, and successful work along fire-proof buildings would require, not only such careful specifications of the steel frame as Mr. Schneider has prepared, but also as careful specifications relating to the fire-proofing. The question of floor girders could be taken as an instance. These are often designed without reference to partitions and without reference to flush, unbroken ceilings. The experience gained through the fire in the Horne Buildings in Pittsburg, and elsewhere, has shown that far better results as regards the fire-proofing are secured from flush ceilings than from paneled ceilings where the girders are allowed to project below the ceiling line. Again, the successful design of spandrel beams or members can only be accomplished by carefully considering the fire-proofing possibilities of the spandrel construction. The successful designer of steel-frame buildings, therefore, should be as familiar as possible with the whole range of fire-proofing, and also as familiar as possible with the architect's standpoint and the problems which he must face. Mr. Freitag.

VIRGIL H. HEWES, M. AM. SOC. C. E.—The different changes of loading which take place after buildings have been erected, and the impossibility of predicting what the changes will be, having been mentioned, the speaker would like to cite a case, which came under his observation while making an examination, of a building in New York City upon which the Building Department had filed a violation. Mr. Hewes.

The plan of the building was of flat-iron type, being wider at one end than at the other, with two sides converging. The walls on one side and on the larger end were supported upon cast-iron columns at the street or sidewalk level; at the smaller end the wall ran down to the foundation. The other side was broken by re-entrant walls to form a light shaft. These walls were also supported on cast-iron columns, while the two sections of wall along the lot line extended down to the foundation.

All the tenants had moved out of the building, except those in one store at the street level, and a printing office on the top floor. The printing office was being moved; everything had been taken out except a large press which was still in place. The pressman, having a piece of work which he wished to turn out before taking down the press, started it up at the highest speed. The table carrying the forms, having a reciprocating motion, caused the building to vibrate, increasing up to a point where it took a gyrating motion, then the motion would die out and then start to vibrate again to the point of maximum vibration.

The speaker was two floors below the printing office when the press was started, and notified the men of the chances they were taking, then he took a vacation till the press was stopped. It is probable that if the lower floors had been loaded, the vibrations would not have been as severe.

This tends to show how necessary it is that the frame of a building should be a structure in itself without depending upon the partitions and walls for stiffness as Mr. Schneider has stated.

Vol. XXX. NOVEMBER, 1904. No. 9.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

A RATIONAL FORM OF STIFFENED SUSPENSION BRIDGE.

Discussion.*

By Theodore Cooper, M. Am. Soc. C. E.

Mr. Cooper. Theodore Cooper, M. Am. Soc. C. E. (by letter).—The writer, in his report to the Quebec Bridge Company, on June 23d, 1899, stated as follows, in reference to the plan described in Mr. Lindenthal's paper:

"The lines of the structure are very pleasing, giving a combined effect of grace and strength. The catenary curves of the cables are not crossed or broken by the stiffening trusses.

"The design appears, from an ordinary examination, to be in accordance with the requirements of the specifications. A stiffening truss of this kind could not be used successfully for bridges formed with continuous wire cables, as the connections of the various members of the truss would have to be made through the frictional grip of cable bands, which would not be trustworthy. The success of such a truss depends therefore upon the use of wire links for the cables and a positive connection of all the members by means of pins. That such links can be made is undoubted, but their successful and economic manufacture has yet to be developed."

At that time the suggestion was made that there might be future cases where eye-bars could be used for this form of bridge.

Engineers, through the daily and technical press, have conveyed the idea that "eye-bar cables" are new and untried forms.

Without detailing the earlier suspension bridges, which were all

*Continued from October, 1904, *Proceedings*. See August, 1904, *Proceedings* for paper on this subject by Gustav Lindenthal, M. Am. Soc. C. E.

link-cable bridges, the fact that the great majority of bridges in North America, over 100-ft. spans, both railroad and highway, are dependent upon "eye-bar cables" for their lower chords, seems to have been overlooked even by some eminent engineers. Mr. Cooper.

The writer would consider it a low estimate to state that there are more than 200 miles of "eye-bar cables" in successful use in the United States, many of which have done excellent duty for a full generation.

The first important structure with which the writer was identified, the St. Louis Arch Bridge, was erected by means of eye-bar cables more than 500 ft. in length, and these same cables, more than 2 600 ft. in total length, are now in use in the Fairmount Bridge, Philadelphia, and in bridges elsewhere.

The "eye-bar cable," forming the top chord of the Quebec Bridge, is now under construction and is formed of eye-bars 15 in. deep.

The eye-bar and the eye-bar cable are the peculiar and especial characteristic of American bridges, and it is rather late in the day for any engineer to discover that they are new and untried forms.

All the large suspension bridges built by the Messrs. Roebling at Niagara, Cincinnati and Brooklyn have eye-bar cables in their anchorages.

In the eye-bar cable engineers know by actual test the strength of the component parts in large units, they know and can allow for the inclinations of the several members forming the cable, and, when erected and under work, they can inspect and see whether or not it is working according to the assumptions.

In the wire cable engineers are asked to have "faith in things unseen" and unproven. They must presume that the several thousand wires forming a large cable are all absolutely parallel after they have been banded together and wrapped, and that every wire is doing equal duty, except within a few inches of the splices.

Engineers must accept the strength of a single wire multiplied by the total number of wires as the strength of the full cable. They must believe that, although all past methods of preserving the inclosed wires have failed to stand the test of time, the modern methods can be relied upon without waiting for a long experience.

For spans up to 2 000 ft. at least, engineers must allow the wire cable a large degree of flexibility, in order that it may enter into competition with the rigid truss under similar conditions of loading, even after accepting the above presumptions.

Until the limit of 2 000 ft. is passed, or the point where the wire-cable bridge may enter into fair competition with the rigid truss, it would seem unnecessary, if not undesirable, to accept a form of bridge with these questionable features, when the same graceful forms can be obtained in a rigid structure formed of large units the strength of

Mr. Cooper. which is known or can be known by actual tests, and which has all its parts open for inspection and preservation.

In the form of stiffened suspension bridge under discussion the strains on each and every member can be determined readily and positively.

The writer does not think the same can be said of the flexible suspension bridge, as heretofore treated. To consider a bridge, where several thousand tons of material are assumed as rising and falling several feet in a few seconds as a structure acted on by statical forces only, appears to the writer to be fallacious.

As to the competitive plans for the Quebec Bridge, the several plans were submitted to the writer for examination, and on his report the contract was let. There were submitted three plans for wire bridges. Two of these plans were by companies also presenting cantilever structures, and they stated that they could get no prices for the wirework which would justify tenders. The third plan, though accompanied by a tender, gave no strain sheets, sizes of parts, or plans of foundations. The schedule of material accompanying the tender gave but little more than half the wire and half the masonry contained in the other plans.

PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. XXX. No. 10.

DECEMBER, 1904.

Edited by the Secretary, under the direction of the Committee on Publications.

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NEW YORK 1904.

American Society of Civil Engineers.

Vol. XXX. DECEMBER, 1904. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

December 7th, 1904.—The meeting was called to order at 8.40 P. M., Rudolph Hering, M. Am. Soc. C. E., in the chair; Chas. Warren Hunt, Secretary; and present, also, 107 members and 24 guests.

The minutes of the meetings of November 2d and 16th, 1904, were approved as printed in the *Proceedings* for November, 1904.

A paper entitled "Probable Wind Pressure Involved in the Wreck of the High Bridge over the Mississippi River, on Smith Avenue, St. Paul, Minn., August 20th, 1904," by C. A. P. Turner, M. Am. Soc. C. E., was presented by the Secretary, who also read a communication on the subject by Theodore Cooper, M. Am. Soc. C. E. The paper was then discussed orally by Messrs. G. E. Gifford, H. W. Brinckerhoff, N. A. Melick, S. Bent Russell, H. F. Dunham, R. A. MacGregor, H. C. Keith, S. T. Wagner and H. P. Macdonald.

The Secretary presented a letter from the American Association for the Advancement of Science, inviting members of the Society to attend the meetings of the Association to be held in Philadelphia at the University of Pennsylvania, December 27th to 30th, 1904.

Ballots for membership were canvassed, and the following candidates elected:

As Members.

William Eugene Austin, New York City.
Job Rockfield Furman, New York City.
William Montgomery Gardner, Memphis, Tenn.
James Noble Hatch, Chicago, Ill.
Emil Louis Nuebling, Reading, Pa.
Richard Harvey Phillips, St. Louis, Mo.
John Horton Pope, New York City.
Harold Ulmer Wallace, Chicago, Ill.
Edgar True Wheeler, Los Angeles, Cal.

As Associate Members.

Fred Asa Barnes, Ithaca, N. Y.
William Hunt Bowne, Glen Cove, N. Y.
Varnum Pierce Curtis, Worcester, Mass.
John Jerome Dalton, Asheville, N. C.
Samuel Edwards Fairchild, Jr., Philadelphia, Pa.
Arthur James Griffin, Brooklyn, N. Y.
Henry Atkinson Holdrege, Omaha, Nebr.
Joseph Prosper Horstman, Parkersburg, W. Va.
Robert Elmer Horton, Utica, N. Y.
George Danforth Huntington, Watertown, N. Y.
Ivar Kreuger, Johnnesburg, South Africa.
George Latimore Lucas, Trenton, N. J.
Ira Welch McConnell, Montrose, Colo.
George Alexander Hutchings Mould, Johannesburg, South Africa.
Herbert Damon Newell, Ontario, Ore.
George Frederick Folger Osborne, Bombay, India.
George Bigelow Pillsbury, Washington, D. C.
Marshall Rogers Pugh, Philadelphia, Pa.
Hugo Julius Scheuermann, Albany, N. Y.
Harry Linden Shaner, Lynchburg, Va.
Herman Franklin Tucker, Kendal Green, Mass.
Willis Tubbs Turner, Salt Lake City, Utah.

FRANK GEORGE WHITE, Salt Lake City, Utah.
JOSEPH WRIGHT, Sheffield, Ill.
GEORGE WASHINGTON ZORN, Cheyenne, Wyo.

AS ASSOCIATES.

THEODORE MARTIN MAY, New York City.
VINCENT IGNATIUS STEPHEN, Monywa, Upper Burma, India.

The Secretary announced:

The transfer of the following candidates, by the Board of Direction, on December 6th, 1904:

FROM ASSOCIATE MEMBER TO MEMBER.

EDWIN STANTON FICKES, Pittsburg, Pa.
SIFROY JOSEPH FORTIN, City of Mexico, Mexico.
ARTHUR WILLARD FRENCH, Worcester, Mass.
JOHN CLARK SPENCER, Pittsburg, Pa.
EVERETT BROOMALL WILSON, Evanston, Ill.

The election of the following candidates, by the Board of Direction, on December 6th, 1904:

AS JUNIORS.

EDWARD CARTWRIGHT CONSTANCE, St. Louis, Mo.
HARRY JOHNSON DEUTSCHBEIN, Albany, N. Y.
WILBUR HOWARD FISHER, Hannibal, Mo.
CHARLES GILMAN, New York City.
WILLIAM LAWRENCE HANAVAN, New York City.
JOHN PHILIP HOGAN, New York City.
HARRY NORTHROP HOWE, Greenville, Miss.
SAMUEL JAMES LEWIS, Montrose, Colo.
CLARENCE ADKINS NEAL, Kansas City, Mo.
JOHN CASTLEREAGH PARKER, Niagara Falls, N. Y.
GEORGE JOHNSON WALKER, Self, Ark.
LEE FIELD WHITBECK, Durango, Dgo., Mexico.

The Secretary announced the following deaths:

FREDERICK REGINALD FRENCH; elected Member September 7th, 1904; died November 20th, 1904.

LORENZO MEDICI JOHNSON; elected Junior March 3d, 1875; Member April 7th, 1880; died November 28th, 1904.

Adjourned.

December 21st, 1904.—The meeting was called to order at 8.45 P. M., Vice-President S. L. F. Deyo in the chair; Chas. Warren Hunt, Secretary; and present, also, 97 members and 13 guests.

A paper, entitled "The Reclamation of River Deltas and Salt Marshes," by J. Francis Le Baron, M. Am. Soc. C. E., was presented by the Secretary, who also read a communication on the subject from E. L. Corthell, M. Am. Soc. C. E.

The paper was discussed by Richard Lamb, M. Am. Soc. C. E.

The Secretary announced the following deaths:

JACOB ALBERT LATCHA, elected Member May 7th, 1873; died November 30th, 1904.

WILSON CROSBY, elected Member September 15th, 1869; died December 18th, 1904.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

December 6th, 1904.—8.35 P. M.—Vice-President Curtis in the Chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Buck, Croes, Deyo, Ellis, Gowen, Knap, N. P. Lewis, Noble, Osgood, Pegram, Webster and Wilgus.

A report, from the Committee on Entertainment of the party of members of the British Institution of Civil Engineers, was received, which showed that the funds of the Society had not been drawn upon for this purpose, and that 55% of the amount subscribed had been returned *pro rata* to the subscribers.

The following resolution was adopted:

"*Resolved*, That the thanks of the Board of Direction of the American Society of Civil Engineers are hereby tendered to Paul A. Seurot, M. Am. Soc. C. E., for his aid in the translation of a number of French papers for the International Engineering Congress."

A Committee was appointed to confer with the Architect and to report plans for the enlargement of the Society House.

A Committee of Arrangements for the Annual Meeting was appointed:

The following resignations were accepted, to take effect December 31st, 1904:

Max Boehmer, M. Am. Soc. C. E.; Duncan Lee Despard, Assoc. M. Am. Soc. C. E.; Edward Betts Brisley, Jun. Am. Soc. C. E.

Applications were considered and other routine business transacted.

Five Associate Members were transferred to the grade of Member, and twelve candidates for Junior were elected.

Adjourned.

ANNOUNCEMENTS.

The House of the Society is open from 9 A. M. to 10 P. M. every day, except Sundays, Fourth of July, Thanksgiving Day and Christmas Day.

MEETINGS.

Wednesday, January 4th, 1905.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper, entitled "Methods of Location on the Choctaw, Oklahoma and Gulf Railroad," by F. Lavis, Assoc. M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

Wednesday, February 1st, 1905.—8.30 P. M.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper entitled "Maximum Rates of Rainfall at Boston," by Charles W. Sherman, M. Am. Soc. C. E., will be presented for discussion.

This paper is printed in this number of *Proceedings.*

ANNUAL MEETING.

The Fifty-second Annual Meeting will be held at the Society House, January 18th and 19th, 1905. The Business Meeting will be called to order at 10 o'clock on Wednesday morning. The Annual Reports will be read, officers for the ensuing year elected, and members of the Nominating Committee appointed.

Arrangements for the excursions and entertainments have been placed in the hands of the following committee:

JOHN W. ELLIS, *Chairman;*
GEORGE S. WEBSTER, CHARLES L. HARRISON,
J. WALDO SMITH, CHAS. WARREN HUNT.

NOMINATING COMMITTEE.

The Constitution provides that at the Annual Meeting of each year, seven Corporate Members, not officers of the Society, one from each of the seven geographical districts, into which the territory occupied by the Membership is divided for this purpose, shall be appointed by the meeting to serve for two years.

The usual blank request for suggestions as to representatives of each district, for presentation to the meeting, has been mailed to Corporate members.

PRIVILEGES OF LOCAL SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

The Boston Society of Civil Engineers will welcome any member of the American Society of Civil Engineers at its library and reading room, 517 Tremont Temple, Boston, which is open on week days from 9 A. M. to 5 P. M. Members will also be welcome at the meetings, which are held in the same building, on the evenings of the fourth Wednesday in January, and the third Wednesdays of other months, except July and August.

The rooms of **The St. Louis Engineers' Club,** are in the business center of St. Louis, and visiting engineers are cordially invited to use them for mail, telephone service, information, etc.

The courtesies of **The Engineers' Society of Western Pennsylvania** have been extended to members of the American Society of Civil Engineers. The rooms of the Society, 410 Penn Ave., Pittsburg, Pa., are open at all times, and meetings are held as follows, except during July and August. REGULAR SECTION, Third Tuesdays; CHEMICAL SECTION, Thursdays following third Tuesdays; MECHANICAL SECTION, First Tuesdays; STRUCTURAL SECTION, Fourth Tuesdays.

The Western Society of Engineers, Monadnock Block, Chicago, Ill., tenders to members of this Society the use of its rooms and facilities, together with the good offices of its Secretary and of a special committee appointed for that purpose.

The Civil Engineers' Club of Cleveland, Ohio, invites members of this Society to make use of the Club rooms, at any time when in Cleveland. Cards will be furnished on application to the Secretary, Mr. J. C. Beardsley.

The Engineers' Club of Central Pennsylvania has established new quarters at the corner of Second and Walnut Streets, Harrisburg, Pa., and desires to extend the courtesies of the Club to visiting members of the American Society of Civil Engineers.

SEARCHES IN THE LIBRARY.

In January, 1902, the Secretary was authorized to make searches in the Library, upon request, and to charge therefor the actual cost to the Society for the extra work required. Since that time many such searches have been made, and bibliographies and other information on special subjects furnished.

The resulting satisfaction, to the members who have made use of the resources of the Society in this manner, has been expressed frequently, and leaves little doubt that, if it were generally known to the membership that such work would be undertaken, many would avail themselves of it.

The cost is trifling, compared with the value of the time of an engineer who looks up such matters himself, and the work can be performed quite as well, and much more quickly, by persons familiar with the Library.

Copies of all lists of references are filed, so that in many cases it is only necessary to make a typewritten copy, which reduces the cost of searches to a minimum.

In asking that such work be undertaken, members should specify clearly the subject to be covered, and whether references to general books only are desired, or whether a complete bibliography, involving search through periodical literature, is desired.

ACCESSIONS TO THE LIBRARY.

November 8th to December 13th, 1904.

DONATIONS.*

FOWLER'S ELECTRICAL YEAR BOOK;

And Directory of Light, Power and Traction Stations, 1905. Boards, 6 x 4 in., 588 pp., illus. Manchester, Scientific Publishing Company. 1 shilling 6 pence, net.

In this edition of the Year Book substantial additions have been made to the several sections dealing with the types and use of electrical measuring instruments. The section on the running of dynamos in parallel, as well as some notes on the management of dynamos, the preface states, will, it is hoped, prove useful. The recommendations of the British Standards Committee for generators and motors have been embodied in the present edition, while additions have been made to the sections dealing with electrical distribution, secondary batteries, and lighting. The extension of the application of alternating current has demanded more extensive treatment, and some additions have been made to the subject of electric traction.

AMERICAN RAILWAY SHOP SYSTEMS.

By Walter G. Berg. M. Am. Soc. C. E. Cloth, 9 x 6 in., 198 pp., illus. New York, *The Railroad Gazette*, 1904.

The aim of the author in presenting this treatise has been to collect, in convenient form for reference, general information as to the layout and leading characteristics of railway repair shops, particular attention being paid to establishing a special grouping and classification of shops as an aid to an intelligent analysis and understanding of the subject. The Contents are: Classification and General Layout; General Repair Shops; Locomotive Repair Shops; Passenger Car Repair Shops; Freight Car Repair Shops; General Shop Store Houses; Power Plant and Machinery; Structural Work of Buildings and Auxiliary Features. The book contains a bibliography of eight pages, and a large number of illustrations.

EXPERIMENTS WITH ALTERNATE CURRENTS OF HIGH POTENTIAL AND HIGH FREQUENCY.

A Lecture Delivered Before the Institution of Electrical Engineers, London; With an Appendix on the Transmission of Electric Energy without Wires, Reviewing his Recent Work, and Presenting Illustrations from Photographs never before Published; With a New Portrait and a Biographical Sketch of the Author. By Nicola Tesla. New Edition, cloth, 7 x 5 in., 9 + 162 pp., illus. New York, McGraw Publishing Company, 1904. $1.00.

Since the year 1890 Mr. Tesla has devoted himself entirely to the study of alternating currents of high frequencies and very high potentials, with which he is at present engaged. The biographical introduction states that no comment is necessary on his interesting achievements in this field. His first lecture on his researches in this new branch of electricity, was delivered before the American Institute of Electrical Engineers on May 20, 1891. The present lecture forms in a measure a continuation of the latter, and includes chiefly the results of his researches since that time.

THE ASSUAN RESERVOIR AND LAKE MOERIS.

A Lecture Delivered at a Meeting of the Khedivial Geographical Society, Cairo, 16th January, 1904; with Translations in French and Arabic. By Sir William Willcocks. Cloth, 10 x 7 in., 36 + 40 + 40 pp., illus. New York, Spon and Chamberlain, 1904. $2.00.

In this lecture the author outlines what is in his opinion the best method to secure additional water for irrigation in Egypt and at the same time obtain protection from floods. He describes the Assuân Dam and the design for raising it and the project for the Wady Rayan Reservoir, or the modern Lake Moeris, and also describes the proposed rectification of the Nile by a system of jetties at the Rosetta mouth of the river.

* Unless otherwise specified, books in this list have been donated by the publisher.

LES RICHESSE MINERALES DE LA NOUVELLE-CALEDONIE.

Rapport a M. le Ministre des Colonies. Par E. Glasser. Paper, 9 x 5 in., 560 pp., illus. Paris, Vve. Ch. Dunod, 1904. 10 francs.

The Minister of the Colonies, for France, charged the author with a mission to New Caledonia, for the purpose of studying its mineral wealth; the results are contained in this book. After giving general information on the geological formation of the different deposits of New Caledonia, the author writes of each mineral in turn, giving its known beds, the manner of working them, the industrial conditions, and of what development they appear susceptible. Nickel, cobalt, chrome, iron, copper, gold, coal, etc., are thus reviewed. The work ends with considerations on the future of the mines of the colony and the measures to be taken to assure their success. The book contains six maps and plates.

Gifts have also been received from the following:

Albany State Engr. and Surv. 3 pam.
Allentown—Water Dept. 1 pam.
Alvord, J. W. 1 pam.
Am. Inst. of Elec. Engrs. 1 bd. vol.
Am. Inst. of Min. Engrs. 2 pam.
Am. Iron and Steel Assoc. 1 pam.
Am. Ry. Assoc. 1 vol.
Am. Soc. of Mech. Engrs. 1 pam.
Am. Water Works Assoc. 1 vol.
Boston—Transit Comm. 1 bd. vol.
Brit. Fire Prevention Committee. 1 vol., 1 pam.
Brooks, Fred. 1 pam.
Burr, W. H. 1 pam.
Cal.—Dept. of Highways. 2 pam.
Canada—Geol. Surv. 1 bd. vol., 1 vol.
Cincinnati—Engr. Dept. 1 pam.
Cincinnati, New Orleans & Texas Pacific Ry. Co. 1 pam.
Colo. Agri. Coll. 2 pam.
Detroit—Board of Water Commrs. 1 pam.
District of Columbia—Engr. Comm. 1 pam.
Fisk, W. L. 1 pam., 2 maps.
Fisk & Robinson. 1 pam.
Francois, Felix. 1 pam.
Great Brit.—Patent Office. 4 vol., 14 pam.
Houston, J. J. L. 1 bd. vol.
Huergo, L. A. 25 pam. 18 vol.
Indian Midland Ry. Co. 1 pam.
Ill.—Bureau of Labor Statistics. 1 bd. vol.
Inst. of Civ. Engrs. 1 bd. vol.
Inst. of Engrs. & Shipbuilders in Scotland. 1 bd. vol.
Inst. of Mech. Engrs. 1 vol.
Kansas City Southern Ry. Co. 1 pam.
Kirchbach, Frank. 1 pam.
Long Island R. R. Co. 1 pam.
La. & Arkansas Ry. Co. 1 pam.
Mich.—State Board of Health. 1 pam.
Municipal Art Soc. of N. Y. 1 pam.
National Electric Light Assoc. 1 bd. vol.
National Mosquito Extermination Society 1 pam.
N. Y.—Rapid Transit R. R. Comm. 1 bd. vol.
North of England Inst. of Min. & Mech. Engrs. 2 pam.
Permanent Inst. Assoc. of Navigation Cong. 1 vol.
Poor's R. R. Manual Co. 1 bd. vol.
Queensland—Harbours & Rivers Dept. 1 pam.
R. Scuola d'Applicazione per gl'Ingegneri. 1 pam.
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Reinforced Concrete. By Charles F. Marsh. New York, D. Van Nostrand Company, 1904.

SUMMARY OF ACCESSIONS.

November 8th to December 13th, 1904.

Donations (including 41 duplicates and 4 numbers of periodicals to complete volumes)	289
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DEATHS.

French, Frederick Reginald........Elected Member, September 7th, 1904; died November 20th, 1904.

Johnson, Lorenzo Medici............Elected Member, April 7th, 1880; died November 28th, 1904.

Latcha, Jacob Albert...............Elected Member, May 7th, 1873; died November 30th, 1904.

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(November 6th to December 10th, 1904.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
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(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Canada.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*. Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Inst., Hoboken, N. J., 50c.
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(11) *Engineering* (London), W. H. Wiley, New York City, 25c.
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(20) *Iron Age*, New York City, 10c.
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(22) *Iron and Coal Trades Review*, London, England, 25c.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gas Light Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Engineer*, New York City, 10c.
(28) *Journal*, New England Water-Works Assoc., Boston, $1.
(29) *Journal*, Society of Arts, London, England, 15c.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des École Spéciales de Gand*, Brussels, Belgium.
(32) *Mémoires et Compte Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Économique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*. Paris, France.
(37) *Revue de Mécanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill., 10c.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Engrs., New York City, 50c.
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(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 50c.
(45) *Mines and Minerals*, Scranton, Pa., 20c.
(46) *Scientific American*, New York City, 8c.
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(54) *Transactions*, Am. Soc. C. E., New York City, $5.
(55) *Transactions*, Am. Soc. M. E., New York City, $10.
(56) *Transactions*, Am. Inst. Min. Engrs., New York City, $5.
(57) *Colliery Guardian*, London, England.
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(63) *Minutes of Proceedings*, Inst. C. E. London, England.
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(77) *Journal*, Inst. Elec. Engrs., London, England.

LIST OF ARTICLES.

Bridge.

The Erection of Iron Bridges.* Russell Scott Scholefield. (63) Vol. 157.
The Breydon Viaduct at Great Yarmouth.* William Marriott and Theodore Graham Gribble, MM. Inst. C. E. (63) Vol. 157.
Ferro-Concrete Bridge.* (12) Oct. 28; (11) Oct. 28.
The Strength and Stability of Stone and Brick Bridges.* (21) Serial beginning Nov.
New Bridge Near Mayence.* (12) Nov. 11.
The Thebes Bridge.* (14) Nov. 12.
Renewing Bridges on the West Shore.* (15) Nov. 18.
The Anatomy of Bridgework.* W. H. Thorpe, Assoc. M. Inst. C. E. (11) Serial beginning Nov. 18.
Repairing a Wrecked Drawbridge over the Maumee River at Toledo.* (14) Nov. 19.
Designs for Bascule Bridges without Tail Pits.* (13) Nov. 24.
The Bridge over the Rhine at Thusis.* (12) Nov. 25.
A Large Sheet Pile Cofferdam.* (14) Nov. 26.
The Calumet River Drawbridge, Baltimore & Ohio R. R.* (14) Nov. 26.
The Erection of the Double-Track Mingo Cantilever Bridge over the Ohio River.* (14) Serial beginning Nov. 26.
Structural Details of the New Reinforced Concrete Bridge at Grand Rapids, Mich.* Wm. F. Tubesing. (13) Dec. 1.
An Interesting Steel Arch Bridge Design.* (13) Dec. 1.
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Concrete Abutment on the Ulster & Delaware.* M. H. McGee. (15) Dec. 2.
Reinforced Concrete Arch Bridges, Como Park, St. Paul.* (14) Dec. 3.
The New Pend d'Oreille Bridge, Northern Pacific Ry.* (18) Dec. 3.
The South Tenth Street Bridge, Pittsburg.* (14) Dec. 10.
Reconstruction of Viaduct over the Michigan Central Railroad, by Michigan Traction Company, near Galesburg, Mich.* (17) Dec. 10.

Electrical.

The Evolution of the Submarine Telegraph.* Charles Bright, Assoc. M. Inst. C. E. (63) Vol. 157.
Some Notes on the Edison Nickel-Iron Storage Battery.* F. M. Davis. (4) Oct.
Thirty-Thousand-Volt Transmission. J. F. Kelly and A. C. Bunker. (Abstract of Paper read before the International Elec. Cong.) (73) Oct. 28.
The Measurement of the Potential of the Electrodes in Stationary Liquids.* Henry J. S. Sands. (Paper read before the Faraday Soc.) (73) Serial beginning Oct. 28.
Notes on Faults in Small Lead-Covered Wires.* Donald Smeaton Munro. (26) Oct. 28.
The Lancashire Dynamo and Motor Co.'s Works at Trafford Park.* (26) Oct. 28; (12) Nov. 18.
The So-Called International Electrical Units. F. A. Wolff. (Abstract of Paper read before the International Elec. Cong.) (26) Oct. 28.
The Superiority of Alternating Current for the Supply of Current to Large Cities. Gotthold Stern. (Abstract of Paper read before the International Elec. Cong.) (73) Oct. 28.
Transformer with a Large Electrostatic Capacity.* (73) Oct. 28.
The Unobtained Wave-Lengths between the Longest Thermal and the Shortest Electric Wave Yet Measured. E. F. Nichols. (Abstract of Paper read before the International Elec. Cong.) (73) Oct. 28.
The Transposition of Electrical Conductors.* Frank F. Fowle. (42) Nov.
The Short-Circuit Current of an Induction Motor.* G. W. Howe. (73) Nov. 4.
Coherer Action.* K. E. Guthe. (Abstract of Paper read before the International Elec. Cong.) (73) Nov. 4.
Electrolytic Receivers in Wireless Telegraphy.* Lee De Forest. (Abstract of Paper read before the International Elec. Cong.) (73) Nov. 4.
The Bay Counties Power Co.'s Transmission System. L. M. Hancock. (Abstract of Paper read before the International Elec. Cong.) (73) Nov. 4.
American Practice in High-Tension Line Construction and Operation. F. A. C. Perrine. (Abstract of Paper read before the International Elec. Cong.) (73) Nov. 4.
Heysham Harbour Electric Power Plant.* (26) Nov. 4.
Economics of a 200-Mile Transmission.* J. Eugene Wallace. (27) Nov. 5.
The Testing of Transformer Iron.* Lancelot W. Wild. (73) Nov. 11.
Theory of Wireless Telegraphy.* John Stone Stone. (Paper read before the International Elec. Cong.) (73) Nov. 11.
The Press-Button Control of Lifts.* (12) Nov. 11.
The Stalybridge, Hyde, Mossley and Dukinfield Tramways and Electricity Board's Scheme.* (26) Nov. 11.
Report on the Wireless Telegraph Communication Between St. Louis and Chicago.* (27) Nov. 12.
The Power Plant of the Lewis Publishing Company, St. Louis, and the Largest Searchlight in the World.* (27) Nov. 12.
Thermo-Electric Receivers for Wireless Telegraphy and Telephony.* (19) Nov. 12.
The Willesden Power House, London, England.* (27) Nov. 12.
The Electrical Equipment of the Karawanken Tunnel.* (27) Nov. 12.

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* Illustrated.

Electrical—(Continued).

The Radcliffe U. D. C. Electricity Works.* (26) Nov. 18.
York Corporation Electricity Works.* (26) Nov. 18.
The Unipolar Dynamo.* J. Seidener. (From *Zeitschrift für Elektrotechnik.*) (26) Nov. 18.
An Application of Telpherage.* (40) Nov. 18.
Methods of Cable Laying.* Leonard Andrews. (73) Nov. 18.
The St. Louis Exhibition (Electrical Exhibits). (11) Nov. 18.
Insulation for High Pressures.* Harris J. Ryan. (Abstract of Paper read before the International Elec. Cong.) (73) Nov. 18.
Telegraphy and Telephony in Japan. Saitaro Oi. (Abstract of Paper read before the International Elec. Cong.) (73) Nov. 18.
Modern High-Speed Printing Telegraph Systems. J. C. Barclay. (Paper presented before the International Elec. Cong.) (19) Nov. 19.
Siemens & Halske Printing Telegraph or Telecryptograph.*. L. Ramakers. (19) Nov. 19.
The Testing of Lightning Rods. (Tr. from *Allgemeine Chemiker Zeitung.*) (19) Nov. 19.
Speed Indicators.* Clarence P. Feldmann. (27) Nov. 19.
An Alternating Current Relay for Low Frequencies.* Frank F. Fowle. (27) Nov. 19.
Latest Improvements in Carbon-Making. Eugene Ornstein. (After a Lecture of T. Zellner.) (26) Nov. 25.
The St. Louis Exhibition: The Exhibits of the Electrical Controller and Supply Company. (11) Nov. 25.
Electric Traveling-Cranes at the St. Louis Exhibition.* (11) Nov. 25.
A Method of Measuring Magnetomotive Forces.* Rudolf Goldschmidt. (73) Nov. 25.
The Alternating-Current Theory of Transmission Speed Over Submarine Telegraph Cables. A. E. Kennelly. (Abstract of Paper read before the St. Louis International Elec. Cong.) (73) Nov. 25.
Starting and Regulating Resistances.* (73) Nov. 25.
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Loads on Factory Motors. (46) Nov. 26.
An Economical Power Plant at Lima, Ohio.* Frank B. Rae. (17) Nov. 26.
Electric Transmission Devices for Automobiles: The Jeantaud and the Electrogenia Systems.* (19) Nov. 26.
The Induction Motor with High Resistance Secondary. C. J. Spencer. (27) Nov. 26.
Lightning Rods, Wireless Telegraphy and the Dispersion of Fogs by Electricity. A. Frederick Collins. (27) Nov. 26.
The Commercial Development of Water Power. Alton D. Adams. (10) Dec.
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Coal Conveying Plant at Stuart Street, Manchester, Power Station.* I. William Chubb. (64) Dec.
Efficiency of Closed Heaters. W. E. Crane. (64) Dec.
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The Combined Lighting and Heating Plant of the Columbus, Ohio, Public Service Company.* (27) Dec. 3.
Special Applications of Electric Motors at the World's Fair.* Cloyd Marshall. (27) Dec. 3.
New Motor Speed Controller.* (27) Dec. 3.
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Niagara Power in Buffalo. (27) Serial beginning Dec. 3.
Electromagnetic Torque. A. S. McAllister. (27) Dec. 3.
On the Experimental Determination of the E. M. F. of Dispersion of Stray Field of an Armature Winding. C. F. Guilbert. (27) Dec. 3.
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The American Electric & Controller Company Rheocrat.* (20) Dec. 8.
The Use of Electricity in Driving Coal-Conveying Machinery.* Frank C. Perkins. (46) Dec. 10.
Sur l'Établissement Rationnel des Moteurs Asynchrones. C. F. Guilbert. (36) Oct. 25.
Travaux d'Installation de l'Usine Hydro-Électrique de la Compagnie Toronto and Niagara Power. (33) Oct. 29.
La Force Motrice à l'Exposition de Saint-Louis: Groupe Electrogène des Sociétés Delaunay-Belleville et l'Éclairage Électrique.* Charles Dantin. (33) Oct. 29.
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Mesure d'un Défaut d'Isolement entre un Cable et la Terre (Réseau en Charge).* J. Vinson. (36) Nov. 25.
La Voiture Mixte "Krieger."* M. Poulain. (36) Nov. 25.

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* Illustrated.

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H. M. S. *King Edward VII.** (12) Nov. 4.
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Steam Closing Stop Valves for Boilers.* A. B. Willits. (20) Nov. 24.
Simple Methods in Warship Design a Necessity.* George W. Dickie. (Abstract of Paper read before the Soc. of Naval Archts. and Marine Engrs.) (13) Nov. 24.
Maintenance of Machinery in Merchant Ships. Robert Haig. (Abstract of Paper read before the Soc. of Naval Archts. and Marine Engrs.) (13) Nov. 24.
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The Development of Dry-Dock Pumping Machinery.* Joseph S. Shultz. (9) Dec.
Recent Experiments on Screw Propellers at the U. S. Model Basin.* D. W. Taylor. (Paper read before the Soc. of Naval Archts. and Marine Engrs.) (13) Dec. 1.
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Notes on Lutes and Cements. Samuel S. Sadtler. (2) Oct.
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Utilization of Exhaust Steam in Connection with Low Pressure Steam Turbines.* Leonce Battu. (4) Oct.
Torsional Vibration of Shafts. Isaac Vincent Robinson, Stud. Inst. C. E. (Abridged.) (63) Vol. 157.
Rotary Kiln Fuel Consumption. P. O. Krottnaurer. (67) Oct.
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A Proposed Modification of the Perfect Heat-Engine Formula.* Louis Illmer, Jr. (8) Nov.
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The Assistance of Superheated Steam for Reducing the Cost of Electrical Energy.* E. J. Fox. (22) Nov. 4.
Shrinkage Troubles and Methods of "Feeding." Thos. D. West. (Paper read before the New England Foundrymen's Assoc.) (47) Nov. 5.
Notes on Emery Wheels. (47) Nov. 5.
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Air in Relation to Boiler Feeds.* James Alexander Smith. (Paper read before the Victorian Inst. of Engrs., Melbourne.) (62) Nov. 10.
The Relation of Calorific Power to Illuminating Power of Gas. (11) Nov. 11.
Hydraulic Forging-Presses.* (11) Nov. 11.
Milling Cutters.* Thomas R. Shaw. (47) Serial beginning Nov. 12.
Heavy Grading with Hydraulic Monitors. C. H. Rollins. (Abstract of Paper read before the Pacific Northwest Soc. of Engrs.) (14) Nov. 12.
Consumers' Meters.* B. H. Spangenberg. (Paper read before the Amer. Gas Light Assoc.) (24) Serial beginning Nov. 14.
A New Machine for Molding Cement and Sand Bricks.* (13) Nov. 17.
The Adriance New Type Combined Drawing and Reducing Press.* (20) Nov. 17.
The Liquid Fuel Investigation of the Navy Department. (62) Nov. 17.
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Pipes and Joints for High Pressures.* Franklin Riffle. (Abstract of Paper read before the Technical Soc. of the Pacific Coast.) (22) Nov. 18.
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Gas Engine Testing at the Works of the Westinghouse Machine Co. E. E. Arnold. (From the *Journal* of the Elec. Club of Pittsburg.) (14) Nov. 19.
The Properties of Blue Gas. H. F. Hills. (24) Nov. 21.
An Ingenious and Effective Air-Lift Pump.* (13) Nov. 24.
Chemistry in the Foundry. R. Moldenke. (Paper read before the New England Foundrymen's Assoc.) (20) Nov. 24.

* Illustrated.

Mechanical—(Continued).

Steam Engines and Turbines. F. H. Bowman. (Lecture before the National Assoc. of Colliery Mgrs.) (22) Nov. 25.
The Coming of the Gas Turbine. E. Kilburn Scott, A. M. I. C. E. (26) Nov. 25.
The Smoke Problem.* F. J. Rowan. (Paper read before the Inst. of Engrs. and Shipbuilders in Scotland.) (47) Serial beginning Nov. 26.
The National Steel Foundry Company.* (14) Nov. 26.
The Revivification of Oxide in Situ. James Bell. (Paper read before the Scottish Junior Gas Assoc.) (24) Nov. 28.
A Gravity Cement Mill in the Pyrenees.* Geo. M. Peek. (9) Dec.
Stocker Cooling Tower.* (9) Dec.
The Panama Automatic Conveyor and Dumping Car.* (41) Dec.
The Jones Underfeed Stoker.* (Rept. of Committee on Science and the Arts.) (3) Dec.
Power Plants of the Atlas Portland Cement Co.* (64) Dec.
Future Progress of the Steam Engine.* Ed. F. Williams. (64) Dec.
Comparison of Boiler Trials.* J. H. Wallace. (64) Dec.
Suction Gas Producers.* B. A. Sinn. (64) Dec.
Lubricating the Valve and Cylinder.* J. W. Parker. (64) Dec.
The Steam Turbines of the St. Louis Exposition.* (64) Dec.
Experiment and Practice in the Construction of Steam Boilers. Egbert P. Watson. (9) Dec.
Power Required by Machine Tools.* (25) Dec.
The Poston Paving Brick Co., Crawfordsville, Ind.* (76) Dec.
Continuous Kilns.* (76) Serial beginning Dec.
The Campus Brick and Tile Works, Campus, Ill.* (76) Dec.
A New Type of Engine.* (17) Nov. 26; (76) Dec.
The Scottdale Corliss Engine.* (64) Dec.; (20) Dec. 8.
Modern Planers.* Joseph Horner. (10) Dec.
Motor Equipped Pump Building Shops.* 20 Dec. 1; (27) Dec. 1.
Profit in the Use of High Speed Steel Tools.* (20) Dec. 1.
The New Prentice 18-Inch Lathe.* (20) Dec. 1.
The Forter Charger for Annealing Furnaces.* (20) Dec. 1.
A Novel Trench Excavating Machine.* (13) Dec. 1.
Power Hoists for Stationary and Trolley Service.* (40) Dec. 2.
"La Magnetique" Automobile: A Simplified Type of Machine with Magnetic Clutches.* (10) Dec. 3.
The Silent Chain for Driving Machinery.* (14) Dec. 3; (17) Dec. 3; (27) Dec. 3.
Gas Engines in the Works of the Power & Mining Machinery Company.* (17) Dec. 3.
Some Foreign Yard Cranes and Derricks.* (18) Dec. 3.
More New Types of Prime-Movers at World's Fair.* (27) Dec. 3.
Rotary Engines.* F. Niethammer. (27) Dec. 3.
Some Large Engines in the Power Plant of the St. Louis Exhibition.* (13) Dec. 8.
The Best Economy of the Piston Steam Engine at the Advent of the Steam Turbine. J. E. Denton, M. Am. Soc. M. E. (Paper read before the International Cong. of Arts and Sciences.) (13) Dec. 8.
The New Warner & Swasey Turret Lathe.* (20) Dec. 8.
The Indiana Volume Blowers and Exhausters.* (20) Dec. 8.
The American Electric & Controller Company Rheocrat.* (20) Dec. 8.
An Automatic Gas Producer.* (16) Dec. 8.
Gas Engines in Power Plants. Wm. T. Magruder. (Paper read before the Ohio Soc. Mech., Elec., and Steam Engrs.) (62) Dec. 8.
New Apparatus for Charging and Discharging Gas Retorts.* Emile Guarini. (19) Dec. 10.
Nouveau Système de Cabestan Electrique.* F. Loppé. (36) Oct. 25.
La Force Motrice a l'Exposition de Saint-Louis: Groupe Électrogène des Sociétés Delaunay-Belleville et l'Éclairage Électrique.* Charles Dantin. (33) Oct. 29.
Niveaux d'Eau pour Chaudières à Vapeur.* (34) Nov.
Machines á Mouler les Tuyaux Cintrés.* (34) Nov.
Graisseurs Mécaniques pour Voitures Automobiles.* (34) Serial beginning Nov.
La Force Motrice a l'Exposition de Saint Louis: Turbine Curtis.* L. Piaud. (33) Nov. 12.
La Force Motrice a l'Exposition de Saint-Louis: Groupe Électrogène de la Société Alsacienne de Constructions Mécaniques.* G. Zweifel. (33) Nov. 19.
Machines à Vapeur à Pistons-Valves de la Nouvelle Station Centrale d'Électricité de Mulhouse.* (33) Nov. 19.

Metallurgical.

Ore and Stone-Crushing Machines.* (11) Oct. 28; (12) Oct. 28; (22) Oct. 28.
Foundry Practice with Copper and its Alloys. W. J. Reardon. (Paper read before the Electric Club.) (47) Oct. 29.
On Small Plants for the Production of Castings. R. M. Daelen. (22) Nov. 4.
Electric Smelting of Iron and Steel. J. H. Stansbie. (Abstract of Paper read before the Staffordshire Iron and Steel Inst.) (22) Nov. 4.
Tube Mills: the Mode of Operation.* Hermann Fischer. (68) Nov. 5; (Abstract from the *Zeitschrift* des Vereines Deutscher Ingenieure.) (16) Nov. 17.

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Roasting at the Tyee Smelter.* E. Jacobs. (16) Nov. 10.
Recent Developments in Crushing and Concentrating Machines. James Thame. (Abstract of Paper read before the Soc. of Engrs.) (47) Nov. 12.
Notes on Southern Pig Iron. Eliot A. Kebler. (Paper read before the Pittsburgh Foundrymen's Assoc.) (20) Nov. 17.
The Iodimetric Determination of Copper. Andrew M. Fairlie. (16) Nov. 17.
Tube Mills Adopted by the Rand Mines. (68) Nov. 19.
American Steel and Wire Company, Cleveland.* (12) Nov. 25.
Automatic Pig-Iron Casting, Cooling and Conveying Plant.* (22) Nov. 25.
Electric Furnace for the Metallurgy of Copper.* (Tr. from *La Nature.*) (19) Nov. 26.
Smelters and Smelting Practice in British Columbia.* William M. Brewer. (9) Dec.
The Molecular Aggregates of Pig Iron and Steel. A. D. Elbers. (24) Dec. 1.
The New Chlorination.* William E. Greenawalt. (16) Dec. 1.
Fan and Positive Pressure Blower Tests. W. H. M'Fadden. (20) Dec. 8.

Military.

Continuous Reading Range and Azimuth Finder; St. Louis Exhibition. (11) Oct. 28.
Disappearing Gun-Carriages.* (19) Nov. 26.

Mining.

Description of Houldsworth Colliery, Dalmellington.* William Smith. (59) Vol. 27, Pt. 1.
An Inburst of Waste-Water at Wallyford Colliery.* Robert Thomas Moore. (59) Vol. 27, Pt. 1.
Fire in a Lanarkshire Colliery, and Description of a Condenser Used Thereat.* James C. Weir. (59) Vol. 27, Pt. 1.
Electric Power Distribution in Mining. T. Huber. (68) Oct. 29.
Power Plants at Collieries. Maurice Deacon, M. Inst. C. E. (From *Second Rept.* of the Royal Commission on Coal Supplies.) (47) Oct. 29.
Small Quick-Running Fans for Mine Ventilation: An Account of Some New Ventilators for this Purpose and the Advantages Claimed.* James Tonge. (45) Nov.
The Hoisting Problem: The Relation of the Underground Requirements, the Engineering and Mechanical, and the Financial Considerations. James R. Thompson. (Paper read before the Lake Superior Mining Inst.) (45) Nov.
Richmond No. 8 Shaft: Methods Employed in Sinking Through Quicksand Near Scranton to Connect with Workings Below.* Frank G. Wolfe. (45) Nov.
Shaft Sinking Through Quicksand: A Description of the Methods Employed at Susquehanna Mine, Hibbing, Minn. H. B. Sturtevant. (From *Proceedings* of Lake Superior Min. Inst.) (45) Nov.
Mechanical Devices at American Collieries.* L. L. Logan. (Abstract of Paper read before the Central Min. Inst. of Western Pennsylvania.) (57) Nov. 4.
Causes of Explosions in Air Compressors. E. Goffe. (Abstract of Paper read before the Mechanical Engineers' Assoc. of the Witwatersrand.) (12) Nov. 11; (14) Dec. 3.
The New Winning at Dawdon Colliery. (57) Nov. 18.
Gibson Patent Electro-Pneumatic Rock Drill.* (26) Nov. 18.
Two Years' Experience in Winning Coal by Machinery. F. O. Kirkup. (Paper read before the National Assoc. of Colliery Mgrs.) (22) Nov. 18.
Notes on the Application of an Oil Engine to Main and Tail and Endless Rope Haulage Combined.* W. C. Blackett. (Paper read before the National Assoc. of Colliery Mgrs.) (22) Nov. 18.
Safety Stop Apparatus for Hoisting Engines.* (68) Nov. 26.
Gold Dredging at Oroville.* Howard D. Smith and Elwyn W. Stebbins. (16) Dec. 8.

Miscellaneous.

The Inch Versus the Metre: For a Universal Metric System. George Moores. (10) Dec.
The Engineering of Industrial Buildings. D. C. Newman Collins. (20) Dec. 1.

Municipal.

The Lighting of Streets in Manchester.* S. L. Pearce. (Rept. of the Special St. Mains Sub-Com. of the Manchester Corporation.) (73) Nov. 4.
Five Years' Operation of the Winnipeg Municipal Asphalt Plant. H. N. Ruttan. (Paper read before the Amer. Soc. of Municipal Impvts.) (14) Nov. 12.
The Oiled Roads of California. (14) Serial beginning Dec. 8.
Bitulithic Pavement in Newark, N. J. (14) Dec. 10.
Le Granit-Asphalte et les Chaussées en Asphalte Armé.* G. Espitallier. (33) Oct. 29.

Railroad.

The Construction of Railway-Wagons in Steel.* James Denis Twinberrow. (63) Vol. 157.
Iron and Steel Railway-Wagons of High Capacity.* James Thomas Jepson. (63) Vol. 157.
The Construction of Iron and Steel Railway-Wagons.* Arthur Lewis Shackleford. (63) Vol. 157.
First-Class Saloon for the South-Eastern and Chatham Railway.* (11) Oct. 28.
North-Eastern Railway Electric Goods Locomotives.* (73) Oct. 28.
Electric Railway with Protected "Live Rail."* L. B. Stillwell. (Abstract of Paper read before the International Elec. Cong.) (47) Oct. 29.

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Standard Rail Sections. (Report of the Engineering Standards Committee.)* (47) Oct. 29; (22) Oct. 28.
The Electric Railway of La Mure.* E. Guarini. (72) Nov.
Cutting Keyways on Driving Axles.* (39) Nov.
Consolidation Freight Locomotive, Canadian Pacific Railway.* (39) Nov.
Hydraulic Piston Puller.* (39) Nov.
Equipment for a Modern Blacksmith Shop for Railroad Work.* A. W. McCaslin. (Paper presented before the National R. R. Master Blacksmiths' Assoc.) (39) Nov.
The Westinghouse Automatic Coupler.* (21) Nov.
Steam Passenger Cars: Taff Vale Railway.* (21) Nov.
Recent Locomotive Work on the London and North-Western Railway. Charles Rous-Marten. (12) Serial beginning Nov. 4.
The Young Rotary Valve and Gear for Locomotives.* (40) Nov. 4; (18) Nov. 5.
The Locomotive and Machine Company of Montreal, Ltd.* (18) Nov. 5.
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The Beard Automatic Car Coupler.* (20) Nov. 10.
French Quadruple-Expansion Engine of 1 500 H. P. at the St. Louis Exhibition.* (13) Nov. 10.
The Washington Terminal Improvements of the Baltimore & Ohio and the Pennsylvania Railways.* W. F. Strouse. (13) Nov. 10.
The Remarkable Progress of the Hudson River Tunnel for the New York & New Jersey R. R. Co.* (13) Nov. 10.
Great Northern Tunnel at Seattle.* (15) Nov. 11.
New Yards and Terminal Approaches at Washington, D. C.* W. F. Strouse. (15) Nov. 11.
The Mallet Articulated Locomotive at the St. Louis Exhibition.* H. W. Hanbury, A. M. I. C. E. (11) Nov. 11.
Auto Cars on French Railways.* (40) Nov. 11.
Rails for Lines with Fast Trains. J. W. Post. (From *Bulletin* of the International Ry. Cong.) (40) Nov. 11.
Boston & Maine 10-wheel Passenger Locomotive.* (40) Nov. 11.
The King-Lawson Dumping Car.* (40) Nov. 11: (25) Dec.
Improvements on the Morris & Essex Division of the Lackawanna R. R.* (14) Serial beginning Nov. 12.
The Steel Construction in the Wabash Terminal at Pittsburg.* (14) Nov. 12.
Public Tests of the New York Central Electric Locomotive.* (46) Nov. 12; (13) Nov. 17; (20) Nov. 17; (15) Nov. 18; (40) Nov. 18; (14) Nov. 19; (27) Nov. 19; (17) Nov. 19; (18) Nov. 19; (25) Dec; (72) Nov.
The Mürren Cable Railroad in the Swiss Alps. (46) Nov. 12.
Rail Lock for Drawbridges, L. S. & M. S. Ry.* (18) Nov. 12.
Baldwin Four-Cylinder Balanced Compound Locomotive.* W. J. McCarroll. (Paper read before the Traveling Engineers' Assoc.) (18) Nov. 12.
Steel Castings. W. A. Herron. (Abstract of Paper presented before the Ry. Club of Pittsburg.) (18) Nov. 12.
How to Maintain Surface, Line and Gage. (Committee Rept. to the Roadmasters and M. of W. Assoc.) (15) Nov. 18.
Bay Shore Cut-Off of the Southern Pacific.* (15) Nov. 18.
Chicago, Burlington & Quincy Line Revision.* (40) Nov. 18.
Safety Appliances and Block Signals: Proposed Bill Requiring Use of the Block System. (Rept. of Committee to the National Assoc. of Ry. Commissioners.) (40) Nov. 18.
The Shops of the Locomotive & Machine Company of Montreal.* (40) Nov. 18; (25) Dec.
Tidewater Terminal of the Western Maryland at Baltimore.* (15) Nov. 18.
Eight-Coupled Locomotive for Goods Traffic; Great Central Railway.* (11) Nov. 18.
Experiments with Manganese Frogs in the Broad Street Terminal, Philadelphia.* (17) Nov. 19.
Boiler Troubles on Large Locomotives. Harry Bently. (Paper presented before the Iowa Ry. Club.) (18) Nov. 19.
German Four-Cylinder Balanced Compound Locomotive, with Superheater, at the St. Louis Exhibition.* (13) Nov. 24.
Comparative Cost of Fast and Slow Freight Service.* W. B. Poland. (From *Bulletin* No. 56 of the Amer. Ry. Eng. and M. of W. Assoc.) (15) Nov. 25; (40) Nov. 25; (18) Nov. 26; (13) Dec. 1.
Steel Cars. J. F. MacEnulty. (Abstract of Paper read before the New England R. R. Club.) (15) Nov. 25.
Improvements at the Spencer Shops of the Southern Railway.* (15) Nov. 25.
The Grand Trunk Pacific. W. L. Mackenzie King. (From the *Quarterly Journal of Economics.*) (15) Nov. 25.
Passenger Stations of the Duluth, South Shore & Atlantic.* (15) Nov. 25.
The New Haven's Merchants' Limited Train.* (40) Nov. 25.
Standard Locomotives for the Southern Pacific.* (40) Nov. 25.
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Draft in Ash Pans.* (Committee Rept. presented before the North-West Ry. Club.) (18) Nov. 26.

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The Railways of Natal: Their Characteristics, Trains and Train Service.* J. F. Gairns. (10) Dec.
Freight Locomotives with Superheaters 2-8-0 Type: The Canadian Pacific Railway.* (25) Dec.
The Actual Building of a Chinese Railway.* Justin Burns. (9) Dec.
Systematization and Tool-Room Practice in a Railway Repair Shop. R. Emerson. (9) Dec.
Powerful Prairie Type Locomotive: Lake Shore & Michigan Southern Railway.* (25) Dec.
Instructive Tests of a Baldwin Balanced Compound Locomotive: Burlington & Missouri River Railroad. (25) Dec.
Vauclain Four-Cylinder Balanced Compound Locomotive: 4-6-0 Type: New York, New Haven & Hartford Railway.* (25) Dec.
Boiler Repairs with Treated Water: Records from Chicago & North Western Railway. (25) Dec.
The Old Monroe-Mexico Branch of the Burlington.* A. W. Newton. (15) Dec. 2.
Norfolk & Western Station at Roanoke.* (15) Dec. 2.
Class "K" Prairie Locomotive for the Lake Shore.* (40) Dec. 2; (15) Dec. 2.
Tie Timber and Spikes in Argentine Republic. (15) Dec. 2.
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Angus Locomotive and Car Shops, Montreal: Canadian Pacific Railway. (25) Serial beginning Dec.; (40) Serial beginning Dec. 9.
The Northern Pacific Mikado Locomotive.* (40) Dec. 9.
Signals at St. Louis Union Station.* (15) Dec. 9.
A Concrete Wall Eleven Years Old.* (15) Dec. 9.
The Block System on the Vandalia Line. John L. Davis. (Abstract of Address before the St. Louis Ry. Club.) (15) Dec. 9.
Electrification of the North-Eastern Railroad of Great Britain.* (19) Dec. 10.
The Foundations for the Yonkers Power House of the New York Central & Hudson River R. R.: The Present Stage of One of the Two Stations to Furnish Electricity for Handling the Central's Train Service near New York.* (14) Dec. 10.
Les Grandes Vitesses et les Chemins de Fer.* M. Von Borries. (Tr. from *Zeitschrift* des Vereines deutscher Ingenieure.) (38) Nov.
Note sur la Nouvelle Gare de Saint-Omer.* M. H. Le Goaster. (38) Nov.
Le Chemin de Fer et le Port de Narvik (Norvège).* Charles Delgobe. (33) Nov. 5.
Le Chemin de Fer de Chamonix.* F. Loppé. (36) Serial beginning Nov. 10.
Tunnels Sous-Marins du Rapid Transit Railroad entre New York et Brooklyn.* (33) Nov. 19.
Voiture Automotrice Pétroléo-Électrique du North Eastern Railway.* (33) Nov. 26.

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Raworth's System of Automatic Regenerative Control for Electric Traction. John I. Hall. (Abstract of Paper read before the Tramways and Light Rys. Assoc.) (73) Oct. 28.
Sterling, Dixon & Eastern Electric Railway.* (72) Nov.
The St. Gall-Speicher-Trogen Electric Railway.* S. Herzog. (72) Nov.
Piping and Power Station Systems. William L. Morris. (72) Serial beginning Nov.
Piping Diagrams. (72) Serial beginning Nov.
Double Deck Cars with Covered Tops: A Review of Current British Practice.* (72) Nov.
Discussion at St. Louis, Sept. 14, 1904, on "Different Methods and Systems of Using Alternating Current in Electric Railway Motors."* (42) Nov.
Automatic Signals for Electric Tramways.* (21) Nov.
Railway Motors for Single-Phase Alternating Current.* Hans Behn-Eschenburg. (11) Nov. 4.
Notes on Permanent Way for Tramways. A. N. Connett. (Abstract of Paper read before the Tramways and Light Rys. Assoc.) (73) Nov. 4; (17) Nov. 26.
The New York Subway Under the East River.* (20) Nov. 10.
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Steam Motor Coach.* (12) Nov. 11.
Wakefield and District Light Railways.* (12) Nov. 11.
The Economics of Electric Tramways in Britain.* Robert H. Smith. (12) Nov. 11.
The Stalybridge, Hyde, Mossley and Dukinfield Tramways and Electricity Board's Scheme.* (26) Nov. 11.
Average Speeds and Speed Allowances. A. R. Fearnley. (Abstract of Paper read before the Assoc. of Tramway and Light Ry. Officials.) (26) Nov. 11.
The Signal System on the Subway Division, Interborough Rapid Transit Co., New York.* (14) Nov. 12.
Do the Rails of Electric Street Railways Return the Current to the Generating Station, or Simply Act as a Ground? Emile Guarini. (19) Nov. 12.
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Field Notes on the Electric Railway Tests.* (17) Nov. 12.
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The Single-Phase Railway Motor. A. S. McAllister. (Abstract of Lecture before the New York Elec. Soc.) (17) Nov. 12; (27) Nov. 12; Abstract of Discussion (27) Nov. 19; (17) Nov. 26.
Concerning Noise in the Operation of Elevated Railways.* (13) Nov. 17.
Permanent Way for Tramways.* Chas. T. Taylor. (Paper read before the Rugby Eng. Soc.) (47) Nov. 19; Abstract (73) Nov. 18.
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The Cleveland & South Western Traction Company's Unique Line Car.* (17) Nov. 26.
The Railway Between Aberdeen and Hoquiam, Washington.* (17) Nov. 26.
The Autographic Test Car.* Albert B. Herrick. (17) Nov. 26.
An Economical Power Plant at Lima, Ohio.* Frank B. Rae. (17) Nov. 26.
Traction Systems of Paris.* Paul Dubois. (27) Nov. 26.
The Winter-Eichberg Single-Phase Railway System. (27) Nov. 26.
The Parkville Concrete-Steel Sub-Station of the Brooklyn Rapid Transit Co.* (14) Nov. 26.
The Wellington, N. Z., Tramways.* (17) Dec. 3.
Train Resistance.* W. J. Davis, Jr. (17) Dec. 3.
Car Wheels. A. N. Banister. (Abstract of Paper read before the Assoc. of Tramway and Light Ry. Officials.) (17) Dec. 3.
The Scioto Valley Traction Company.* (17) Dec. 3; (14) Dec. 3.
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Mechanical Features of the Twin City Rapid Transit Company's New Power Plant.* Howard S. Knowlton. (14) Dec. 10.
Electric Railways at Green Bay, Wis.* (17) Dec. 10.
Division Terminal Improvements in New Jersey: New Shop Methods.* (17) Dec. 10.
The 1 400-Volt Direct-Current Railway between Tabor and Bechyne, Bohemia.* (17) Dec. 10.
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Des Conditions de la Traction Électrique sur les Chemins de Fer Metropolitains et des Systèmes Dits à Unités Multiples.* Maurice Soubrier. (36) Serial beginning Nov. 25.

Sanitary.

The Cleaning and Flushing of Sewers: Discussion by the Sanitary Section of the Boston Society of Civil Engineers.* (1) Oct.
The Disposal of Municipal Refuse. F. K. Rhines. (1) Oct.
Some Examples of Recent Main Sewer Construction by the City of Philadelphia.* C. H. Ott. (2) Oct.
The Warming of Public Buildings by the Warm-Air System Considered from a Hygienic Point of View. H. H. Grundy. (Paper read before the British Inst. of Heat. and Vent. Engrs.) (70) Nov.
Hot Water Heating Systems with Accelerated Circulation (Theory and Discussion).* G. Debesson. (70) Nov.
A Heating System with Construction Details, Illustrated and Explained.* J. P. Lisk. (70) Serial beginning Nov.
The Sewage Testing Station at Columbus, Ohio.* John H. Gregory. (14) Nov. 19.
The Shone System of Fan Ventilation of Sewers at Darley Abbey and at Leicester, England. (13) Nov. 24.
Sewage Disposal Experiments at Hanley, England. (14) Nov. 26.
Concrete for Brooklyn Sewers. J. C. Brackenridge, M. Am. Soc. C. E. (14) Nov. 26.
Central Station Heating. W. H. Schott. (60) Dec.
Tools Used in Cleaning Sewers.* Ben. E. Briggs. (60) Dec.
The Steam Turbine in a Refuse Destruction Plant. Alfred Gradenwitz. (60) Dec.
Sewage Purification Works for Small Populations. A. J. Martin. (From *The Public Health Engineer.*) (13) Dec. 1.
Sewer Siphons in Brooklyn, N. Y.* (14) Dec. 3.
The Combined Lighting and Heating Plant of the Columbus, Ohio, Public Service Company.* (27) Dec. 3.
The Septic Tanks at Greensboro, N. C. W. G. Potter. (14) Dec. 10.

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Fireproof Floor Construction.* J. T. Montgomery. (4) Oct.
Standard Floor Arch Construction.* E. V. Johnson. (4) Oct.
A Rapid Method for the Determination of Lime in Cement. Bernard Enright. (Paper read before the Amer. Chem. Soc.) (67) Oct.

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Tensile Impact Tests of Metals.* W. Kendrick Hatt. (Paper presented before the Amer. Soc. for Testing Materials.) (47) Serial beginning Oct. 29.
Reinforced Concrete Construction.* John O. De Wolf. (72) Nov.
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Use of Steel in Modern Buildings. John Goodman, M. Inst. C. E. (Lecture delivered before the Bradford Engineering Soc.) (47) Nov. 12.
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The Erection of the New York Public Library.* (14) Nov. 12.
Two New Timber Preservation Processes at the St. Louis Exhibition.* (13) Nov. 17.
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Moving and Erecting Large Granite Columns.* (14) Nov. 19.
The Steel Framing of the New Amsterdam Theatre, New York.* (14) Serial beginning Nov. 19.
Concealed Underpinning for a New York Residence.* (14) Nov. 26.
A Fire Test of a Concrete Floor.* Ira W. Woolson. (14) Nov. 26.
Steel Concrete Chimneys.* (64) Dec.
Natural Hydraulic Cements. E. C. Eckel. (60) Dec.
Strength of Reinforced Concrete Beams. (60) Dec.
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The Erection of the Trinity Building, New York.* (14) Dec. 3.
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The Works of the Power and Mining Machinery Co.* (14) Dec. 3.
Six Story Reinforced-Concrete Factory Building in Brooklyn.* (14) Dec. 10.
The Constitution of Portland Cement from a Physico-Chemical Standpoint. Clifford Richardson. (Address before the Assoc. of Portland Cement Manufacturers.) (19) Serial beginning Dec. 10.
Le Platre.* J. Malette. (36) Oct. 25.
Note sur un Procédé Graphique pour Déterminer la Pression Maxima à la Base d'un Mur.* P. Busset-Schiller. (36) Oct. 25.
Le Liège Aggloméré dans la Construction. M. Boulat. (36) Oct. 25.
Cheminées d'Usines en Ciment Armé.* (35) Nov.
Expériences et Théories de M. C. Richardson sur la Constitution du Ciment de Portland. E. Lemaire. (33) Nov. 19.
Étude sur les Matériaux Réfractaires. Marie-Auguste Morel. (36) Nov. 25.

Topographical.

How the Massachusetts Northern Boundary was Run Without Random Lines. N. Spofford. (14) Dec. 10.

Water Supply.

The Forms of Turbines most suitable for Low Falls.* Alphonse Steiger, M. Inst. C. E (63) Vol. 157.
Methods for the Removal of Incrustation in Water-Mains; with a Description of the Automatic Machine Employed in the Melbourne Water-Supply.* Edgar Gowar Ritchie. (63) Vol. 157.
Irrigation in the Native Reserves of the Colony of Natal.* Albert James Humby, M. Inst. C. E. (63) Vol. 157.
Thawing Services and Mains by Electricity. J. M. Diven. (Paper read before the Amer. Water Works Assoc.) (70) Nov.
The Odessa Water Works.* (11) Nov. 4.
The Governing of Impulse Water-Wheels.* John Goodman, M. Inst. C. E. (11) Nov. 4.
The Prevention of Lead Poisoning from Water Service Pipes at Teplitz-Schönau, Bohemia. (13) Nov. 10.
Experiments and Formula for the Efficiency of Tangential Water-Wheels.* B. F. Groat. (13) Nov. 10.
Hydraulic Relief Valves for Water-Wheel Penstocks.* (13) Nov. 10; (27) Nov. 12.
The Coolgardie Water Works.* (19) Nov. 12.
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Vol. XXX. DECEMBER, 1904. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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Vol. XXX. DECEMBER, 1904. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

METHODS OF LOCATION ON THE CHOCTAW, OKLAHOMA AND GULF RAILROAD.

By F. Lavis, Assoc. M. Am. Soc. C. E.

To be Presented January 4th, 1905.

There has been much written on the theory of railroad location, but the writer recalls little on the actual practice and modern methods of procedure in the field, and while he feels that there are many engineers far better qualified than he is to take up the subject, the fact remains that they have not done so, and there is little public record of current practice. It is hoped that this paper will evoke some discussion, which will cover the subject from the many different standpoints and form a basis for more uniform methods.

The writer has been connected with surveys for the location of railroads in many parts of the United States and in South and Central America, during the past fifteen years, and has been impressed by the wide variation in methods adopted by different men and railroad companies, and, to a great extent, by their general assurance that the methods of each were right, and, having had all kinds of assistance and equipment, from almost nothing up, was, therefore, all the more

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

ready to appreciate the conditions under which the surveys were conducted which form the basis of this paper.

During 1902 the writer was engaged in making location surveys for the Choctaw, Oklahoma and Gulf Railroad (now part of the Rock Island System) in Oklahoma, Indian Territory and Northern Texas. F. A. Molitor, M. Am. Soc. C. E., was Chief Engineer, and E. J. Beard, M. Am. Soc. C. E., Principal Assistant Engineer, and to them the writer is largely indebted for much of the matter contained in this paper.

Many different lines were investigated, and, between 1898 and 1902, some 800 miles of branches and extensions of this road were built, scattered through five different states and territories. At times as many as ten or more locating parties were in the field at the same time; the methods adopted, therefore, were such as were adapted to maintaining parties in the field continuously.

The writer has confined himself entirely to the methods of making surveys, and the organization and equipment of parties, as practiced generally on this road, with whatever added notes from his own experience he has thought useful, and has not attempted to consider in any way the theory of railroad location.

It is obvious, of course, that the equipment of field parties must vary with the locality in which they are engaged. It is the general practice, however, except in a very few of the Northeastern States, to provide the parties with a camp outfit more or less complete, according to the policy of the road, its financial status, and the facilities of transport. It is the writer's experience that the completeness of the survey is very apt to vary directly with the completeness of the outfit supplied by the railroad, and he has, therefore, entered more or less minutely into details of camp equipment; for, although this has been noticed elsewhere, particularly in "Rules for Locating Engineers on the Northern Pacific Railroad," by E. H. McHenry, M. Am. Soc. C. E., the practice varies considerably.

The following is a list of the camp equipment furnished by the Choctaw, Oklahoma and Gulf Railroad:

1 Office tent with fly......... 14 by 16 ft.
3 Tents........................ 14 by 16 ft.
1 Cook tent.................. 16 by 20 ft.
3 Drafting and office tables.
½ dozen camp chairs.
Stationery and map chest with necessary stationery, blank forms, drawing paper, etc.

Dining Table:

3 dozen agate ware dinner plates.
3 " " " cups.
2 " " " saucers.
2½ " steel knives.
2½ " " forks.
2½ " German silver teaspoons.
1½ " " " dessert spoons.
1 " " " tablespoons.
½ " tin salt boxes.
½ dozen tin pepper boxes.
½ " round agate ware pans, 2 qt.
½ " " " " " 1 "
1 " " " " " 1 pt.
1 carving knife and fork.
7 yds. oilcloth, 48 in. wide.
3 standard trestles (see sketch, Fig. 3).
5 boards, 12 by 1½ in. by 18 ft. (dressed).

Cooking Utensils:

1 No. 8, 6-hole, wrought-iron range.
1 tea-kettle.
1 large cast-iron pot.
1 small " " "
2 large frying pans.
1 small " pan.
2 griddles.
4 tin pans with covers, 1 gal. each.
2 stewpans.
1 3-gal. coffee-pot.
1 gal. teapot.
4 dripping-pans.
6 baking tins for bread.
12 tin pie plates.
2 butcher knives.
1 steel.
2 large meat forks.
1 chopping-knife.
1 meat saw.
2 large iron spoons.
1 soup-ladle.
1 cake turner.
1 flour sieve.
1 colander.
1 5-gal. tin dishpan.
1 5-gal. " bread pan with cover.
1 chopping-bowl.
1 bread board.
1 rolling-pin.
1 biscuit cutter.
1 nutmeg grater.
1 coffee-mill.
1 spring balance.
6 galvanized-iron buckets.
6 tin dippers (one for each tent and two in cook tent).
2 can openers.
1 corkscrew.
1 broom.
1 scrubbing-brush.
1 alarm clock.
1 table (same as drafting tables).

Miscellaneous:

½ dozen Dietz lanterns.
3 large tin lamps (central-draft, round wicks).
2 large galvanized-iron washtubs.
1 washboard.
4 Sibley stoves (4 lengths of pipe with dampers, 12 lengths of plain pipe) (see Fig. 1).
2 water kegs, 2 gal. each.
6 washbasins.

Tools:

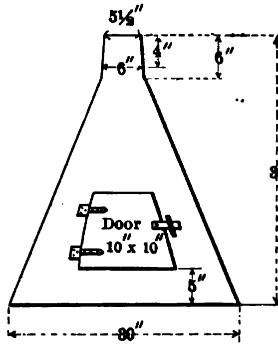
1 grindstone and fittings.
1 monkey wrench.
1 pick.
2 shovels.
1 short crowbar.
1 hand-saw.
1 cross-cut saw.
2 hand-axes.
4 chopping-axes.
½ dozen axe handles.
1 bundle sail twine.
½ dozen sail needles.
1 sail palm.
10 lb. assorted sizes wire nails.
100 ft. manila rope, ¾-in.

Lunch Box:* (See Sketch, Fig. 5).

2 dozen agate-ware dinner plates.
2 " " " saucers.
1½ " steel knives.
1½ " " forks.
1½ dozen German silver teaspoons.
1½ " " " dessert spoons.
1 2-gal. coffee-pot.

* On the Choctaw, Oklahoma and Gulf Railroad, this extra equipment for the lunch box was not ordinarily furnished; the writer, as explained later, believes it to be economy, however, to provide this.

Tents.—The tents furnished were of 12-oz. duck, roped on the seams and ridges with $\frac{3}{4}$-in. Manila rope. They were without ridge poles, four upright poles supporting the center, and four on each side supporting the walls. Tackle was, also, provided, and two single blocks on the front guy rope, there being rectangular door flaps at each end, with substantial leather buckles for fastening them. Leather stove pipe holes with asbestos filling between the leather were provided. The office tent had 5-ft. walls; the others, 4-ft.



SIBLEY STOVE.
No. 18 U.S.S. (0.05")Sheet Steel.
5 Joints Stove Pipe.
Damper in 1st Joint.

FIG. 1.

The writer believes that, when the genuine Mt. Vernon army duck can be obtained, 10-oz. duck gives practically as good service, as far as life is concerned, as 12-oz.; the stiffer duck, when folded, easily cuts and wears in the creases when carried in the wagons. In very hot weather, or in a very rainy country, a 12-oz. fly is desirable. In cold weather, with the lighter tents, in sizes above 14 by 16 ft., it is difficult to heat a tent of 10-oz. duck with the ordinary Sibley stove, but, if necessary to provide camp equipment in a cold climate, the whole equipment can be kept down to that size.

Drafting Tables.—The tops of the drafting tables (Fig. 2) were of $\frac{7}{8}$-in. clear white pine, with hinged legs, connected by 3-in. webbing, arranged so that the legs folded flat against the tops. When moving camp, the tables were placed face to face and tied together, thus preventing injury to the tops.

Dining Table.—The planks forming the top and seats of the dining table (Fig. 3) are placed in the bottom of the wagon when moving camp, as they take up very little room in the bed of the wagon, and the projection of the planks at the rear provides support for the stacked Sibley stoves and other light equipment. The legs of the lower portion of the horses are so spaced as to straddle the wagon and drop down between the bed and the rear wheels.

Stationery and Map Chest.—It is important that this chest (Fig. 4) should be well and strongly made. The protection of the maps, etc., often the results of the expenditure of thousands of dollars, should not depend on any cheap or temporary expedient, as is often the case. It is not an uncommon experience to be caught in the rain while moving camp, and the best of tents are not always impervious to water. It should be insisted upon that all maps, notebooks, etc., should be placed in the chest over night. Necessary stationery, drawing paper, supplies, etc., will vary with different requirements and individual

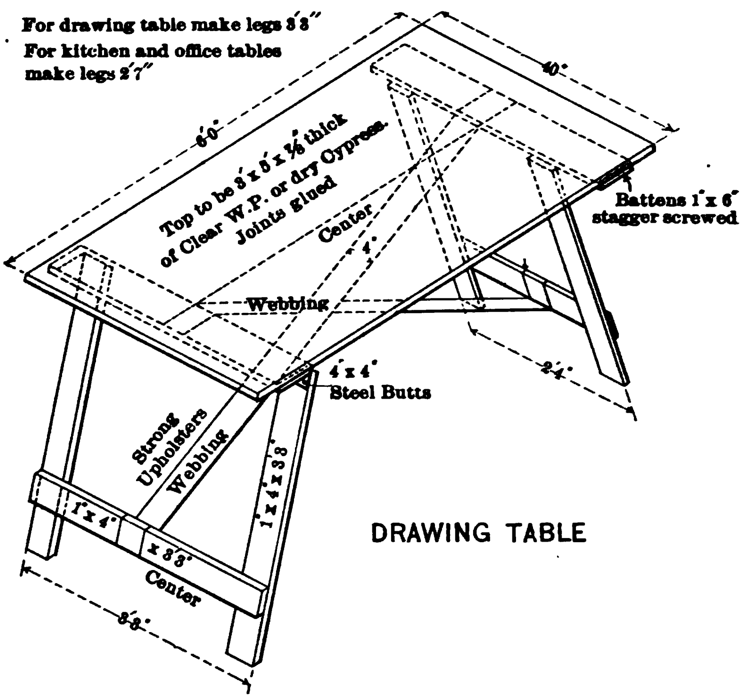


FIG. 2.

preferences. The list given by Mr. McHenry, in the book referred to, is quite complete.

Lunch Box.—The midday meal being eaten in the field, a substantial lunch box (Fig. 5) should be provided, with a separate equipment of plates, knives, forks, etc., from that used in camp.

The party should be ready to start for work immediately after breakfast, and should not be kept waiting while the breakfast dishes are being washed to go into the lunch box, nor should they have to wait for their supper, while the dishes used on the line during the day are being prepared for use. The lunch box can often be best designed in camp after starting, so that it can be made to fit the supplies purchased.

On being organized, the party with which the writer was connected proceeded by rail to the point nearest the proposed line, at which place teams had already been engaged. The cooking utensils and a preliminary bill of supplies had to be purchased, however, that and the loading of the teams occupying the remainder of the day of arrival.

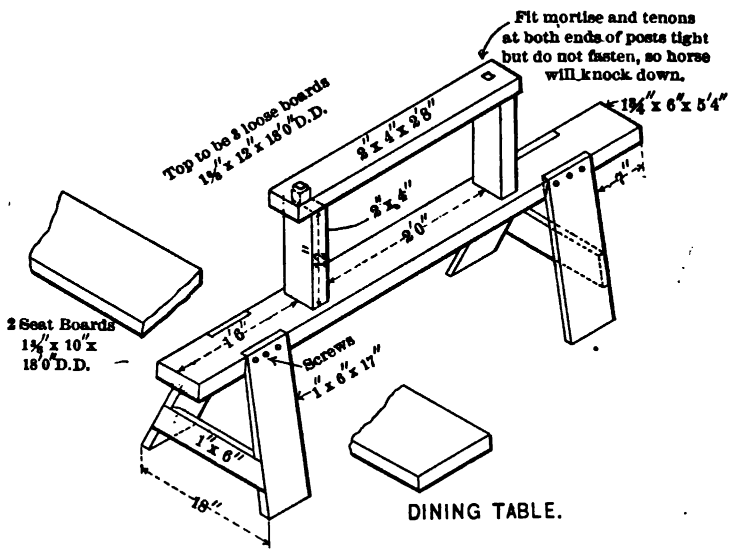


FIG. 8.

The following day at 6 A. M. the outfit was started. Most of the men walked, about one-fourth of them at a time being allowed to ride. Thirty miles over poor roads were covered by 5 P. M., a camping place was selected, tents put up and the men were eating supper by 7 P. M. The first stake was driven before 8 A. M. the next morning, and the work fairly started. This is not noted as an uncommon occurrence, but as representative practice.

In winter the men were called by the cook at 6 A. M. Breakfast was ready at 6.30, and a start was made for the work at 7 A. M.; in summer, half an hour earlier. The teamsters were called and had their breakfast half an hour before the other men, so that they had their teams hitched up and ready to start as soon as the men had breakfasted. Two teams were used on the line, one staying with the topographers. The third team was kept busy keeping up subsistence, supplies, fuel, etc. It has been found more economical, and generally as satisfactory, to employ constantly only two teams in

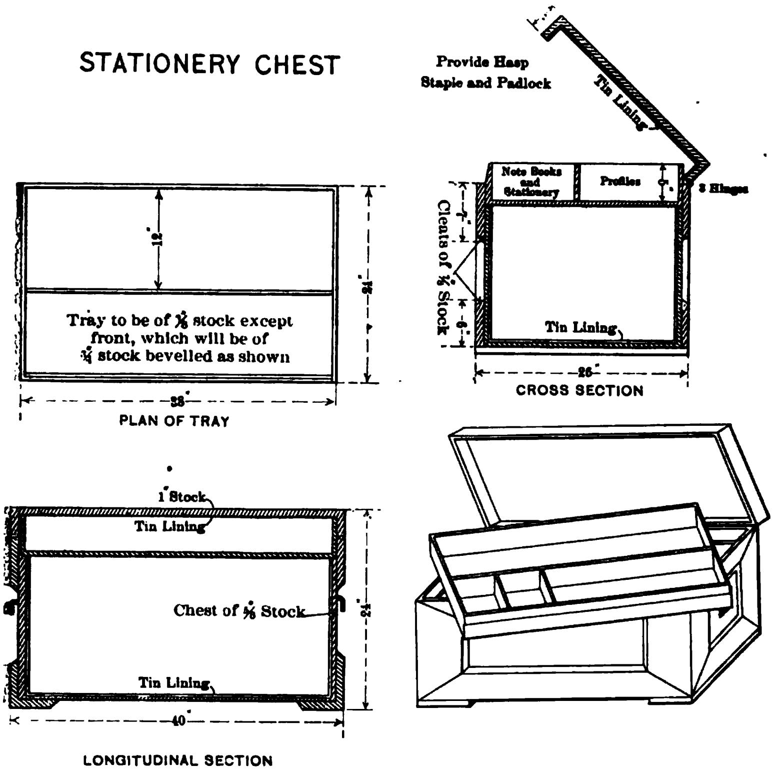


FIG. 4.

settled country where supplies can be easily obtained, and where, on moving days, additional teams can be readily hired.

When moving camp, breakfast was served an hour earlier than usual, the men in each tent then packed up their own things and got their own tent down. Certain men were then assigned to the office tent, and others to the cook tent, this latter with the cook's supplies being the last loaded and first unloaded and put up. A start was usually made by 7 A. M. or a little after, and, with fairly decent roads, about 12 miles, the usual distance between camps, was covered, and the camp up by 2 P. M. The remainder of the day was spent by the party in checking estimates and in various office work, making stakes, getting firewood, etc. If necessary, some arrangement was generally made to keep the topographers and leveler working in the field on days when camp was moved, as they usually had some work to do in order to catch up.

Each locating party was organized as follows:

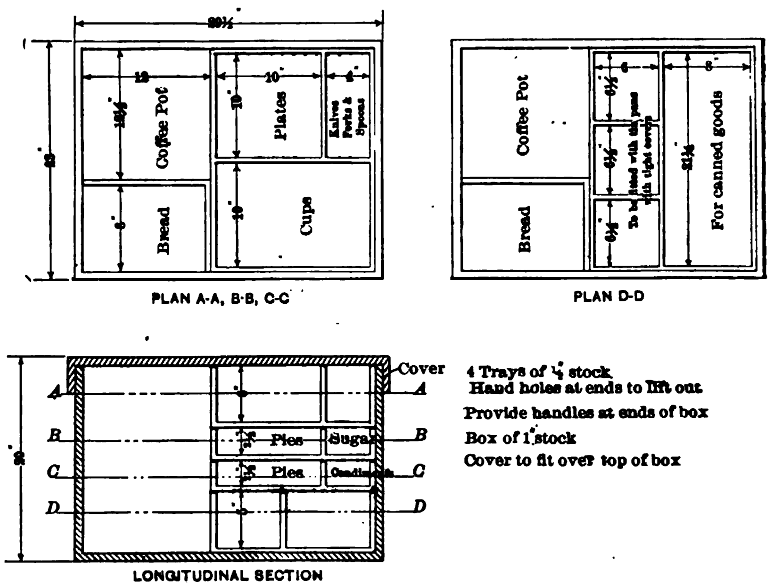
Locating Engineer	$150	to	$175
Assistant Locating Engineer	115	"	125
Transitman	90	"	100
Leveler	80	"	90
Draftsman	80	"	90
Topographers, two*	80	"	90
Rodman	50		
Head Chainman	50		
Rear Chainman	40		
Tapemen, two*	30		
Back Flagman	30		
Stake Marker	30		
Axemen (three to five as necessary)	25	to	30
Cook	50		
Cook's helper	20		
Double teams and driver, furnish their own feed, driver boarded in camp	65	to	90

Each man was supplied by the company with subsistence when in camp, but was required to provide himself with an army cot and sufficient bedding, and advised to provide a substantial canvas covering for the latter, an ordinary wagon cover, costing from $3 to $5, being the most easily obtainable and most satisfactory. The writer has always insisted, as far as possible, that men should equip themselves properly before starting out. The army cot takes up less space than any other cot, both when in use and when folded, and, if the bedding is properly protected, much cause for grumbling is removed on account of its becoming wet or dirty in moving camp, or through the tents' leaking slightly, as the best will do at times.

* See note in regard to topographers, on page 886.

Each man's baggage, besides bedding, was limited to about what could be carried in an ordinary suit case; with the large party and equipment it was found that three good double teams had all they could do to move the outfit, and when the roads were bad it was sometimes found necessary to use an extra team.

Each wagon was required to be provided with a heavy canvas cover and at least one spring seat. The prices for teams varied with the locality and season of the year.



LUNCH BOX

FIG. 5.

The Locating Engineer was provided with a saddle-horse by the company, and an arrangement was made with the head teamster to feed and care for it.

Much of the success of an engineer in charge of a locating party is due to his ability to handle readily the different characters that go to make up the party. Discipline, tempered with judgment, is of course essential, and a certain amount of formality is necessary. Seats were assigned at the table to each man in the order of his rank, and men were required to occupy their proper seats. Conversation was in no way restricted, provided it was gentlemanly, except that no comments, either of praise or blame, upon the food on the table, were permitted. If any one had complaints they were required to make them to the writer out of hearing of the cook, and, on no account, were the men permitted in the cook tent except at meal times.

Frequent inspection of the living tents was made, and it was insisted that each man should make his bed and leave all his things in order before going to breakfast. The men assigned to the living tents were expected to divide between them the necessary chores, required to keep the tent in a clean and tidy condition. In sparsely settled country, considerable attention of this kind is necessary to insure the cleanliness and health of each man; the lazy habits of one should not be allowed to cause unnecessary discomfort to others.

Locating engineers reported directly to the Principal Assistant Engineer. A running account with each was opened with the head office of the railroad, cash being advanced from time to time on requisitions, properly O. K.'d. Provisions were bought by the Locating Engineer at the most convenient points, receipts being taken for all amounts paid out, and an expense account, on proper blanks, with the receipts attached, was sent in at the end of each month, the amounts of which, after being examined and O. K.'d by the Chief Engineer's office, being credited to the account at the head office.

It was found best, as far as possible, to establish a credit with one or perhaps two grocers, preferably wholesale dealers, near the proposed route, and deal with them exclusively, buying all provisions in case lots, or unbroken packages, where possible, thus getting wholesale rates.

It was expected that the men would be provided with good wholesome, plain food of the best quality obtainable. As a general rule, fresh meat and vegetables were difficult to obtain, and canned vegetables, dried fruit, and for meats, ham and bacon, had to be relied on to a great extent. While, of course, the final and principal object of the locating engineer is the location of a line of railroad, the fact should not be lost sight of, that good food and enough of it, properly cooked, is a very important factor in keeping up that *esprit de corps* which is absolutely essential to any degree of success in this particular work.

As stated above, the railroad company expected the men to be properly provided for, but, at the same time, locating engineers were expected to see that there was no undue waste. The experience of the writer was that the expense for provisions for such a party as noted above should be between $250 and $300 per month, and any material increase over the latter amount should be questioned. In many instances, with his and other parties, it was found that a change of cooks often resulted in a large diminution in the expense account, with no difference in the quality or quantity of food provided. It was often found that the better cook was apt to be the more economical. Men to whom problems of the commissariat were new often bought supplies in too small quantities, thus paying as much as 10% more than was necessary.

The expenses of the parties, as noted in the tables of cost on page 894, show costs varying from $220 to $250 per month for provisions alone; the writer, however, has never made a conspicuous record for economy in the matter of buying provisions, opinions on what is proper and what is luxurious varying greatly; but, as with other things, the conditions of each case govern that case, and a happy medium should be striven for. Where the whole party is made up of well-educated and trained professional men, with the exception of the axemen, higher standards of living will be expected. Engineers on such surveys average, for week after week, 100 to 110 hours per week of actual work, as contrasted with 45 to 48 hours for the ordinary business man or office engineer, and the extra cost of providing them with whatever comforts can be reasonably obtained ought not to be objected to.

The expense accounts were carefully examined each month as sent in, and long experience and the many men in the field enabled the assistants at headquarters to determine very closely the reason for any particular account being above the average, and thus call the attention of the locating engineer to the cause in a definite manner.

The following is a list of groceries actually bought on starting a camp:

6 hams.	100 lb. flour, soft wheat.
6 pieces bacon.	100 " sugar.
50 lb. fresh beef.	5 " baking powder.
1 case eggs.	2 " tea.
25 lb. butter.	50 " coffee.
25 " lard.	50 " navy beans.
100 " flour, hard wheat.	25 " Lima beams.

12 lb. buckwheat flour.
5 " macaroni.
35 " cornmeal.
1 cheese (about 15 lb.).
12 packages oatmeal.
10 lb. rice.
100 cakes soap.
1 gal. molasses.
1 case condensed milk.
1 dozen tomato catsup.
¼ " Worcestershire sauce.
1 gal. pickles.
¼ dozen lemon extract.
¼ dozen vanilla extract.
1 box dried prunes.
5 lb. raisins.
4 dozen assorted canned fruits.
1 case tomatoes.
1 " corn.
1 bushel potatoes.
1 kit salt mackerel.
20 lb. salt.
¼ " mustard.
1 " pepper.
1 quart vinegar.
½ dozen yeast cakes.

Most of the lines to which the following methods most closely apply ran through a rather badly broken up, rolling country, with short cross-drainage, about three-quarters being wooded, and, in Indian Territory, very sparsely inhabited.

The low grades desired, 0.5 and 0.6%, compensated for curvature, and the nature of the country involved considerable study of a rather wide range on either side of the proposed general direction of the line. No Government topographical maps of the country had been issued, and the only maps available were those published by the Public Lands Survey, showing fairly accurately, in relation to the section lines, the general location of the larger streams and rivers and some of the main roads.

On a proposed line of about 300 miles in length, a general route was established from previous reconnaissances with certain towns as governing points. Five locating parties were placed in the field, each assigned to about 60 miles. The results desired to be obtained on the location were:

First.—To establish the fact that a practical line could be obtained with ruling grades of 0.6%, or if not, what was the lowest practical ruling grade that could be obtained;

Second.—To be sure that the line obtained was such that no other line could be built through the same country with the same or better ruling grades, with less expenditure, at the same unit prices;

Third.—To keep close control of the work and results of all the parties from a central headquarters;

Fourth.—To have, on the completion of the survey, complete right-of-way maps, estimates of quantities and cost, profiles showing in detail the exact nature of the work, so that contractors could bid intelligently, and work be started at once if necessary;

Fifth.—To keep the cost of the surveys as low as possible consistent with obtaining the above results.

As stated, it was first desired to establish the fact that a reasonable line could be obtained with maximum grades of 0.6% throughout the entire route. Each locating engineer, therefore, was instructed to get one line through as soon as possible on which this condition was established, simply noting carefully, but leaving for future investigation other and perhaps better lines or deviations from the line first run.

It is advisable, where possible, for the Locating Engineer to take a trip over the proposed line before the party gets into the field, but this cannot always be done, as in the particular case referred to. A reconnaissance by the Principal Assistant Engineer, however, had indicated the location of the first camping place and the first work to be done. The party, therefore, was immediately set to work under the direction of the Assistant Locating Engineer, while the Locating Engineer made a more careful reconnaissance over that part of the line assigned him, making the round trip of approximately 120 miles in 4 days.

The trip was made in an open spring wagon; observations with hand-level and compass were taken, and a full sketch was made of the road, showing all branches from it, stream crossings, houses, of which there were few, sketched topography and all the local names that could be learned. Distances were estimated very closely by observing the time of passing all points which could be identified, such as section corners, houses, fences, streams, etc. Fig. 6 shows a page of the notebook used, and Fig. 7 a portion of the completed map.

On this trip, at the end of the line, which was at a town of moderate size, arrangements were made with a local grocer and provision dealer, so that, when it became more convenient to send to that end for supplies than to the town nearest the starting point, all that would be necessary would be to send the teamster with the necessary order.

In working out the details of the location, nearly the whole time of the Locating Engineer was spent looking up the general broad features of the country, the actual work of looking after the field party and running the preliminary line devolving practically entirely on the Assistant Locating Engineer.

In this connection the writer would call attention to a recent article on "Railroad Reconnaissance" in *The Railway and Engineering Review*,* by Willard Beahan, M. Am. Soc. C. E., which describes quite fully and well the work required of the engineer in charge of location, both on the preliminary reconnaissance and in the conduct of the work in the field.

TYPICAL PAGE OF RECONNAISSANCE NOTE BOOK.

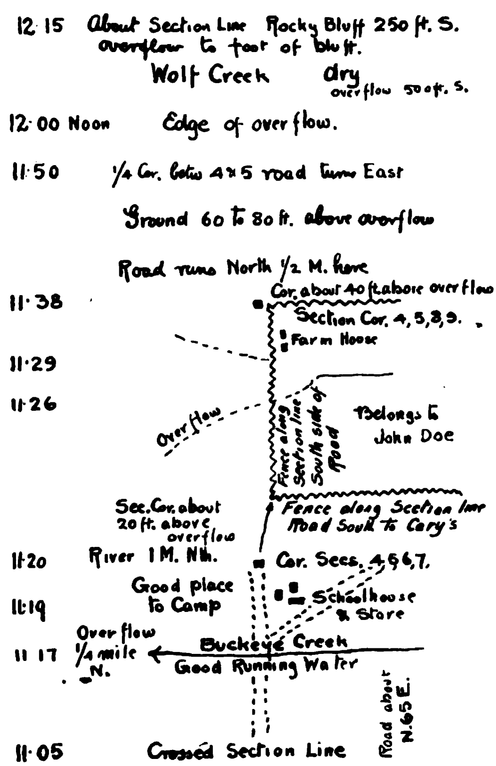


FIG. 6.

In laying out the work for the Assistant Locating Engineer, considerable use was made of rough sketches showing the general topography of the country, and, each evening, with the aid of these and the map and profiles of the lines already run, the following day's

* March 12th, 1904.

work was arranged. During the day, the Locating Engineer found the party at more or less frequent intervals, saw that the line was being run as desired, and made such modifications or revisions of instructions as might be necessary.

It is deemed by the writer most important that the Locating Engineer should be absolutely free at all times, either to stay in camp and keep the office work up to date, to make reconnaissances for any distance ahead, to be with the party at critical points, in fact, to be wholly in an executive position and not tied down to any details. A conscientious study of the country will keep a good man very fully occupied with this work, with "office hours" from 6 A. M. to 9 P. M., and, by working overtime occasionally and planning the work carefully, he will be able to get a couple of hours sometimes on Sundays to write home to his friends.

In the field, the Assistant Locating Engineer had general charge of the party, and, on preliminary work, was responsible for the proper working out of the details of the line from the general location determined on by the Locating Engineer. A special point was made of actually running a preliminary line, getting the topography on it and making a projected location and profile wherever there was a reasonable possibility of a good line existing. Especially is this important in wooded country, where the difficulties of comparatively small details are apt to be magnified, and a good location missed because of the actual physical difficulties of getting over the ground, or of sizing up the topography as a whole. In this connection, it is believed by the writer that little time should be wasted in the field in getting the preliminary so close to where the final location may come that it will show up a good profile. The point of prime importance is to get the preliminary close enough so that the projected location will fall well within the limits of the topography. Of course, the difference between good and bad judgment will show here, as everywhere, but the point is, not to have the party sitting around waiting, and going back to refine the small details on the preliminary; all this can be done to much better advantage on the map, where a broad general view of the whole line can be taken.

On making the final location, the Assistant Locating Engineer obtained from the map the necessary data to connect the located line with the preliminary, and to lay it out on the ground as projected,

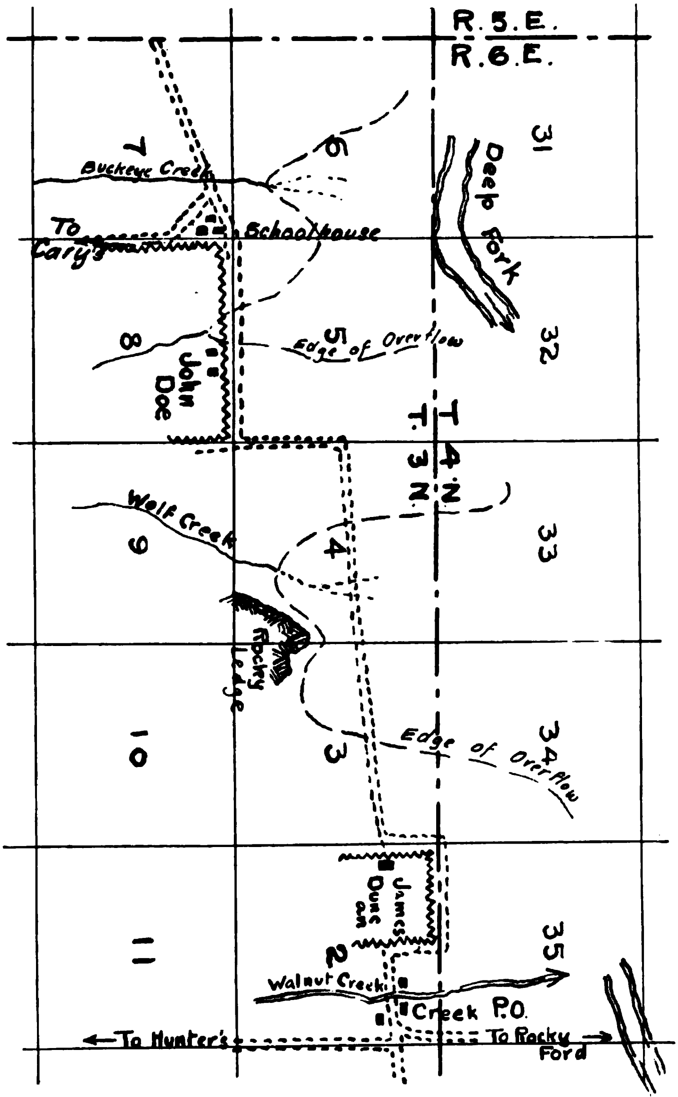


FIG. 7.

FINISHED MAP AS DRAWN FROM NOTES AS SHOWN IN FIG. 6.

which data he kept in a notebook used for that purpose only. He had with him in the field a copy of that portion of the projected profile covering the work in hand. The points on the located line were fixed by horizontal distances from the hubs on the preliminary, and also, and especially where the slopes were at all steep, by vertical distances, and points of elevation fixed by the leveler.

It should be noted especially that, in anything like rough country, the essential point, in running in a location from a line projected on a map, is to reproduce the projected profile rather than the projected line. After deciding that the projected profile is the one to be obtained, the location must be passed through all the controlling points whose vertical distance (or, rather, elevation) is such as called for by the projected profile and determined by leveling from some known elevation (a hand-level will generally give close-enough results).

The actual profile obtained by the leveler was platted in the field at critical points and compared with the projected profile to see that the proper results were being obtained. Absolutely correct topography is not required and is not essential, if the vertical method is adhered to in laying down the location. Many engineers have objected to topographical work on a railroad location on account of the expense necessary for accurate topography, but this latter, and the expense incident to it is entirely unnecessary if the locating engineer in charge of the party knows how to place his projection on the ground, so as to equalize any slight errors made.

The difference between theory and experience, here, is that theory would require an absolutely accurate preliminary and topography, while experience shows that neither is essential, if proper methods are adopted in getting the line on the ground. The writer believes that many locating engineers lose sight of the fact that the location should bear a certain relation of position to the local topography, rather than a relation to a certain geographical position on the map.

It was considered advisable to avoid equalization stations, as the result of revisions of the line, as much as possible, and every care was taken to get the line right in the first place. As the preliminary lines form the basis on which, not only the projection of the location depends, but also of the topographical map, which forms the basis of the final completed record of the survey, it is essential that, without wasting time and money on unnecessary refinements of accuracy, all due precautions should be taken to prevent unnecessary errors.

PRELIMINARY LINES. TRANSIT NOTES.

Line "S"

	Sta.	Dist.	Defl.	Course	Calculated Course
⊙	61 + 40				
⊙	58				
		1020		S.53.00 E.	S.53.56 E.
△	51 + 20		2.00 L.		
		402		S.52.00 E.	S.51.56 E.
△	47 + 18		5.30 R.		
		368		S.57.00 E.	S.57.26 E.
△	43 + 50		4.00 R.		
⊙	36 + 20				
⊙	29				
		1950		S.61.00 E.	S.61.26 E.
△	24		3.00 L.		
		400		S.58.00 E.	S.58.26 E.
△	20		7.00 L.		
		2000		S.51.30 E.	S.51.26 E.
⊙	12 + 31				
		6140			
·	= 0 "S"		21.15 R.		
△	446 + 90 R.				
		970		S.72.30 E.	S.72.41 E.
△	437 + 20 "R" Backsight				

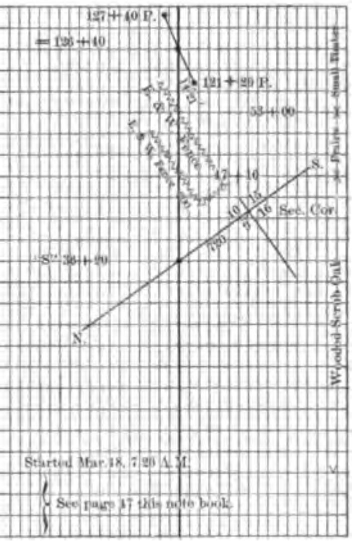


FIG. 8.

FINAL LOCATION. TRANSIT NOTES.

	Sta.	Dist.	Defl.	Course	Cal. C.
△	755 + 19.21	P.C.3° L.			
⊙	751 + 38				
⊙	748 + 25				
⊙	743 + 20				
⊙	742				
		1681.27		N.42.30 E.	N.42.46 E.
△	738 + 37.94	P.T.	5°14′00″		
	738		4.51.14	I.	10°28′
	737		3.51.14	Rad.	2864.93
△	736		2.51.14	S.T.	262.40
	735	523.33	1.51.14	L.C.	523.33
	734		0.51.14		
△	733 + 14.61	P.C.2° R.			
⊙	729 + 50				
⊙	723				
		1276.40		N.32.10 E.	N.32.16 E.
⊙	720 + 38.21	P.T.			

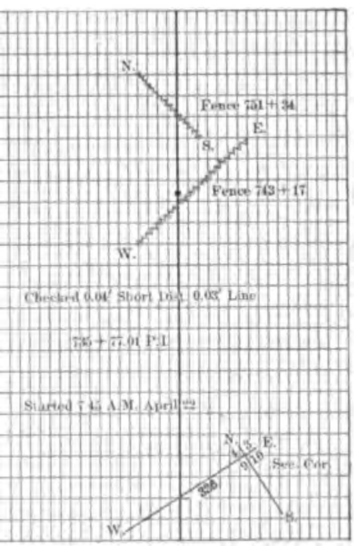


FIG. 9.

All angles were checked by doubling and by compass readings, the rear flagman kept a list of all hubs to check the transitman, and the rodman worked out in a separate book all elevations of turning points, thus checking the leveler, this latter being most important.

In settled cultivated country, on both preliminary and location lines, hubs were set at all fences, as both stakes and hubs in the fields are frequently, and, in fact, usually, pulled up or destroyed soon after the line is run, while the hubs at the fences remain, enabling the ready re-establishment of the line. All tangents were run to an intersection, the P. C. and P. T. of all curves being set before the curves were run in.

Surveys were made of, and levels run on, all railroads crossed, for one mile on either side of the line.

An exact copy of the transit notes was made every night in an office notebook. Figs. 8 and 9 show typical pages of transit notes on preliminary and final location.

The head chainman is responsible to a greater degree perhaps than any other one man in the party for the progress of the work, that is to say, the actual physical progress, after the direction of the line has been determined. He must be a hustler from start to finish and for 10 or 11 hours per day, and, at the same time, be accurate; it is no use being careful about the instrumental work and the platting, if the chaining is not well done, and nothing is more annoying than to plat side lines forming traverses and find that they will not close, or to run in curves on location and not have them check. Chainmen, as a rule, are not college graduates, but are as often picked up in the country where the survey is carried on, and probably have had some experience, and there is no point where the engineer in charge of surveys can use his time to better advantage at the beginning of the work than in getting the chainman to do good work, not necessarily measuring to thousandths, but accurately enough for the work to be done.

There were three axemen (except in one or two cases of very heavy timber for several miles, where two extra men were put on for a short time); in wooded country, all chopped, but in that more or less open, one man helped the stake-marker by driving the stakes.

The writer has found it advisable to pick out one axeman and place him in charge of the others, spending time enough with him at the beginning of the survey to break him in thoroughly to the idea of keeping on line and cutting only what is necessary to get the line through; the right kind of a man will soon learn what is required, and save a great deal of time.

Leveling.—Bench-marks were established every mile on preliminary lines and every half-mile on final location, care being taken to have these latter convenient for construction purposes, especially near bridge openings, etc., and away from all danger of being interfered with by the excavation. The elevation of each bench-mark was plainly marked on or near the same.

TYPICAL PAGE. LEVEL NOTES.

				Line "S"		
Sta.	B.S.	F.S.	H.I.	Rod	Elev.	March 22, Started
					749.20	B.M. 20 ft. L. of 429 + 40 R. on White Oak
	10.24		759.44			N. Bk. 14 page 89
S.. 0				6.2	53.2	
1				8.3	51.1	
+35				10.4	49.0	
+40				10.8	48.6	
2				8.4	51.0	
3				4.1	55.3	
T.P.		1.06			755.28	On large boulder 3 R. of 3 + 45
	7.48		765.86			
4				5.4	60.5	
+65				1.1	64.8	
5				3.2	62.7	
6				5.1	60.8	
+30				7.8	58.1	
+60				10.1	55.8	
7				9.3	56.6	
8				8.1	57.8	
9				6.2	59.7	
T.P.		6.23			759.63	Peg at Sta. 9
	9.54		769.17			769.17 749.20
10				6.4	62.8	19.97
	27.26 7.29 19.97	7.29				

Fig. 10.

Level notes were kept in the form shown in Fig. 10.

On the final location at all bridge openings ravine sections were taken, all plusses being measured in with a cloth tape, and a very

Note.—In using the metric system, stakes are driven every 20 m., and numbered from the beginning, 0, 2, 4, 6, 8, 10, etc. Curves are run in, the same as when using 100-ft. stations, except that the deflections are for 20 m. instead of 100 ft.; as 20 m. equals very closely $\frac{2}{3}$ of 100 ft., the radius of an 8° curve metric equals approximately a 12° using 100-ft. chords. Tables of radii of metric curves are given in Henck and elsewhere.

Levels are taken as usual, ordinary readings to centimeters and target readings to millimeters. Profiles are platted on metric profile paper, the smallest divisions being millimeters, giving a profile, at the scale ordinarily used, about the same as that platted on Plate B paper.

Maps are usually platted on a scale of 1: 5 000, about equal to 400 ft. to 1 in.

careful profile was obtained which was afterward platted on a scale of 10 ft. to 1 in. (see Fig. 11).

Topography.—This was taken for 300 ft. on each side of the line, in cross-section books, ruled, on a scale of 8 ft. to 1 in., in 100-ft. squares, subdivided into 10-ft. squares.

In the field a hand-level and a light wooden rod, 2 in. by ¾ in. by 12 ft., marked every ½ ft., were used, and distances out were paced, or measured with a cloth tape, according to the nature of the ground, and 5-ft. contours were located and sketched in, care being taken at angle points to get sufficient information to connect the contours properly on the map.

As previously noted, absolutely correct topography was not required, and distances were not taped except in cases of very steep slopes; the accuracy with which distances up to 500 ft. can be paced, by any one accustomed to doing it, and levels carried the same distance by hand-leveling, was well brought out in an account of the survey of the Biltmore Estate, by J. L. Howard, M. Am. Soc. C. E.,* and the writer's experience confirms that of Mr. Howard.

Each day the books used on the previous day were left in camp, and the work platted by the draftsman, and other books were taken out. Each book was indexed each night, and each day's work dated at beginning and end.

On the final location, a sounding party was organized, with Topographer No. 1 in charge, and two or three laborers; soundings were taken in all the cuts and at bridge openings, ship augers and steel drills being used most generally. The former were welded to a 12-ft. steel rod with an adjustable handle. These soundings, taken with augers, determined very closely the character of the material in the cuts. In country where boulders might be likely to be encountered to any great extent, either in a clay or gravel formation, this method would not answer; but through the country traversed, as proved on construction, the estimates made from the results of these borings and a close study of the country were quite near the final estimate.

In one or two instances, of important structures and very deep cuts, a regular well-drilling outfit was secured, and the work looked after by a man engaged for the purpose under the direction of the Locating Engineer.

* *Journal of the Association of Engineering Societies*, Vol. XVIII.

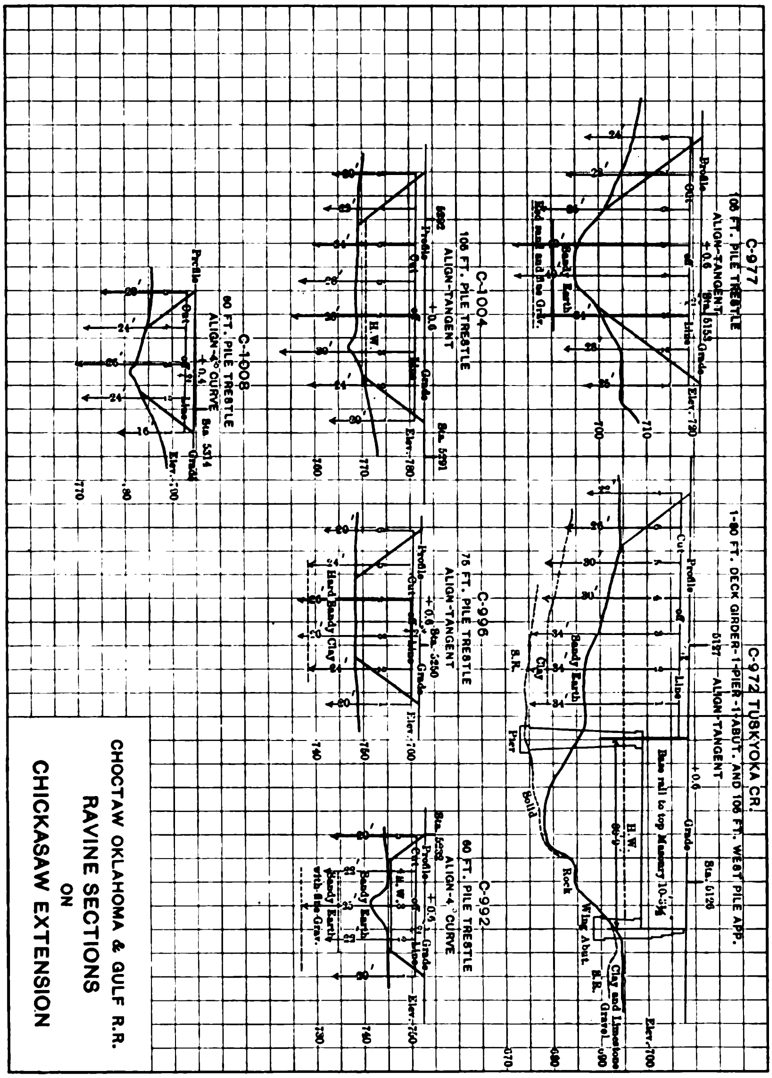

C-977
106 FT. PILE TRESTLE
ALIGN-TANGENT
C-1004
106 FT. PILE TRESTLE
ALIGN-TANGENT
C-1008
60 FT. PILE TRESTLE
C-972 TUSKYOKA CR.
1-80 FT. DECK GIRDER-1-PIER-1-ABUT. AND 106 FT. WEST PILE APP.
ALIGN-TANGENT
C-996
75 FT. PILE TRESTLE
ALIGN-TANGENT
C-992
60 FT. PILE TRESTLE
CHOCTAW OKLAHOMA & GULF R.R.
RAVINE SECTIONS
ON
CHICKASAW EXTENSION

FIG. 11.

The second topographer, supplied with a transit and assisted by the two tapemen, determined the drainage areas, located the property lines and section corners, got names of property owners, etc. This method was found much more economical than to have the whole transit party held up while the transitman and chainmen were getting this information.

With the information thus obtained by the two topographers, the profiles and map of the final location, which were finished within a few days of the completion of the survey, contained all the information necessary to proceed with the construction.

It should be noted here that there were exceptional circumstances in connection with this survey which made it desirable to employ two topographers. Ordinarily, one is sufficient, and a good man will easily take 80% of the topography. Generally, about moving day, the topographer is a day or two behind, in which case the whole party is broken up into topographical parties, and the work cleaned up to the end of the line in a part of a day. Also, when only one topographer is available, when the final location is run in, the Assistant Locating Engineer is occupied about two-thirds of the time in getting land lines, drainage areas, etc., and assisting with the office work, while the Locating Engineer looks after the actual running in of the line. This is ordinarily the most economical arrangement, but, in the survey referred to, it was necessary to rush the work, regardless of the slight extra expense of using men occasionally at a disadvantage.

In carrying out the third requirement of keeping headquarters in touch with the work, a weekly report was made by the Locating Engineer, and the following maps, etc., were kept in shape and up to date:

On preliminary lines: General map, scale 5 000 ft. to 1 in., at the bottom of which was a condensed profile of the projected location, scales 1 000 and 100; detail map, scale, 400 ft. to 1 in.; profiles of preliminary lines and profile of projected location, Plate A paper, scales 400 and 20; profile of projected location on tracing profile paper.

On final location: Line inked in on 400-ft. map, and drainage area shown; right-of-way map, scale 2 000 ft. to 1 in. (required only in Indian Territory); maps of station grounds, scale 100 ft. to 1 in.; final profile on Plate A paper; final profile on tracing profile paper, in 10-mile sections.

Ravine Sections of all Bridge Sites.—The first duty of the field draftsman was the preparation of the general map on the 5 000-ft.-to-1-in. scale, from the best available sources, covering the whole of the country in which the proposed line might lie. In most of the country in the West, where topographical maps have not yet been prepared, the Government maps of the Public Lands Surveys, showing the section, township and county lines, town sites, and the location of the main drainage, will form the basis of this map.

This 5 000-ft. map and profile are absolutely essential to a broad comprehensive study of the line as a whole; it can be readily seen from this whether or not a good general direction is being maintained, and the general relation of the line to the surrounding country is shown. Such a map, with the omission of the preliminary lines, is of considerable aid to contractors in computing the haul of construction material and for other uses; it is also generally sufficient to accompany such reports as are made to the higher officials, in fact, it gives them a more comprehensive idea of the line than a more detailed map. Plate XLVIII shows a portion of a 5 000-ft. map, but shows the located line only; the writer regrets that he has not available a map as described, showing also the preliminary lines and condensed profile.

A tracing was made of this map and, as soon as completed, sent to headquarters; from day to day, the preliminary lines run were platted on it, and, also, the projected location and profile, as they were made.

At the end of each week a tracing of the portion of the map showing the additions made to it during the previous week was sent to headquarters, where the information was transferred to the original tracing.

The weekly report which accompanied this map explained in quite full detail such points in connection with it and the work as seemed to require explanation. It explained, in particular, the natural features of the country, the availability or otherwise of timber (especially for ties), stone, sand, etc., the condition of roads, water supply, and, in general, the work of the party during the preceding week.

All the preliminary lines run during the day were platted on the 400-ft.-to-1-in. map in the evening, from the calculated courses and distances; no platting from deflection angles was allowed. The work of the draftsman was checked by the Assistant Locating Engineer.

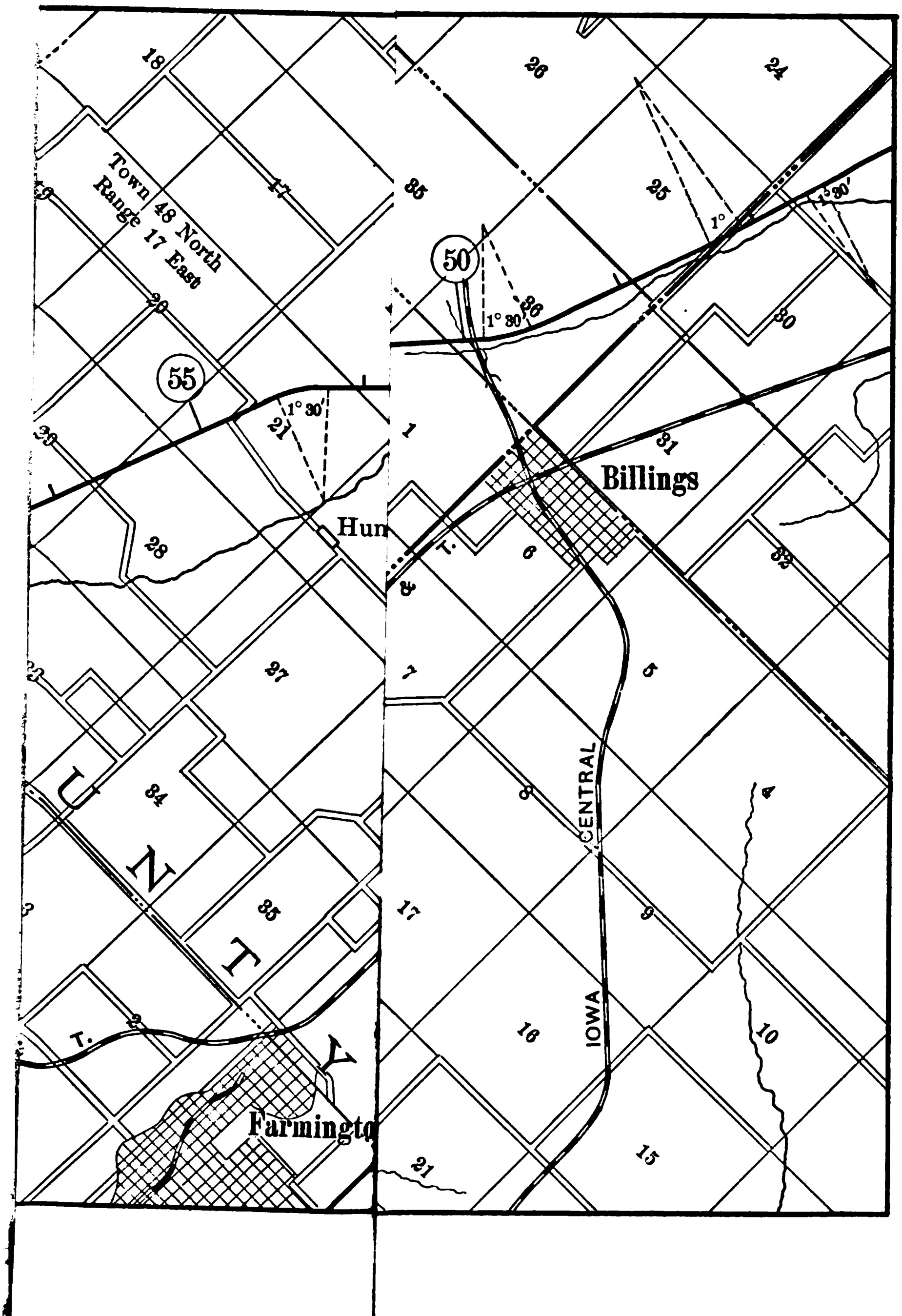
The first thing the following day, the line was inked in, in red, station numbers were marked at all angle points, the topography taken the previous day was platted and inked in immediately in black. Black was found preferable for the contours, as it stood the erasing of the projected lines better than colors. Valleys were indicated by a light blue line drawn through the lowest points, thus making the topography stand out and easier to read. It was found necessary at times to have one topographer stay in half a day and assist the draftsman by platting the topography, in order to keep all the parts of the work co-ordinated.

With the large party used, more than 4 miles per day of preliminary were averaged, and it was absolutely necessary to keep the topography close up, in order that the projected location should not get behind. As a rule, as the map was being used in the evening and the early part of the day, the Locating Engineer endeavored to get back to camp about 4 P. M. and get the projected location up to date before supper. Separate sheets, as advocated by Mr. Wellington in his "Economic Theory," may have some advantage in this respect, but the writer prefers the map on a roll of 36-in. paper. There are few places in difficult country where it is not necessary to run more than one line, and it has always seemed to the writer to detract from a general, broad, comprehensive study of the lines, as a whole, to have them scattered around on separate small sheets of paper.

Necessarily, also, by the separate sheet method, the lines and topography must start right from the edge on one side of the sheet, where it is very likely to be torn; on the 36-in. rolls used on this survey, no topography was allowed within 6 in. of the edge of the paper, except possibly at a point where it just came near the edge and immediately receded from it.

It was absolutely required that the projected location should fall within the limits of the topography, that is, within 300 ft. of the preliminary line, and if, for any considerable distance, it was more than 200 ft. from the preliminary line, it was often deemed advisable to cover this with another line.

In making the projected location, a sheet of tracing cloth, on which was drawn in ink to the scale of the map the curves proposed to be used, with tangents at the ends, was found very useful in fitting the alignment; 100-ft. stations were marked on these curves on the tracing


Town 48 North
Range 17 East
Billings
Farmingt
Hun
CENTRAL
IOWA
55
50
1° 30'
1°
U
N
T
Y

cloth, so that, when laid above the topography sheet, pieces of profile could be readily read off. No P. C. or P. T. was allowed to come within 400 ft. of the end of a bridge; all curves were of even degree of curvature, being either 1, 2, 3 or 4°, and all grades were in even tenths of 1 per cent.

The writer objected to the limitations of the degrees of curve at first, believing that a nicer adjustment could be made by using any degree, with indices of odd minutes when necessary, that seemed at first to fit the ground better, but by being compelled to use the even degrees, found afterward that this could be done almost invariably just as well, but required perhaps more study of the situation. Of course, no rules of this kind can be absolutely iron-clad, and the Principal Assistant Engineer at times modified them himself, but it was considered advisable to make them binding as far as the locating engineers were concerned.

The projected location being made and penciled in on the 400-ft. map, the profile was taken off and the grade line fixed, grades being kept in even tenths of 1%, except where compensated for curvature, and even then, if possible. Of course, in long stretches of ruling grade where often every inch counts, hundredths of 1% rates occurred where compensation was made. Breaks for compensation were made at the even stations nearest the ends of the curves.

All bridges and culverts were located on this profile, the probable quantity of classified excavation in each cut was indicated, and an estimate made of each mile. The classification of the material in the cuts, as shown on this projected profile, was made by the Locating Engineer from his observations of the surface indications; of course, this was only approximate, but was quite close.

Specifications and standard plans of all structures, with tables of constant quantities, were furnished by the railroad, and the excavation and embankment quantities were figured from tables of level cuttings for the standard roadbed sections used. A useful device for scaling the quantities from the profile was made by taking a piece of the same profile paper used for the profile and marking, along the edge at each foot, the quantities corresponding to the height, starting at 0 (see Fig. 12).

As each 10 miles of this profile of the projected location was completed, a tracing on tracing profile paper was made, showing the estimated quantities of each mile and, also, on a regular estimate blank, a summary of quantities, and a summary showing:

Total length,
" degrees of curve,
" length of tangent,
" percentage of line on curve,
Maximum curve,
" grade,
Total rise (in the direction of the line),
" fall (in the direction of the line),
" length of bridging,
" cost,
Average cost per mile,
" number of cubic yards per mile.

These tracings of 10 miles each, with the estimates and summary, were forwarded to headquarters as fast as completed, and, on the completion of the line, an estimate and statement similar to the above, covering the whole line, was sent in. The original profile was made in 25 to 30 mile sections.

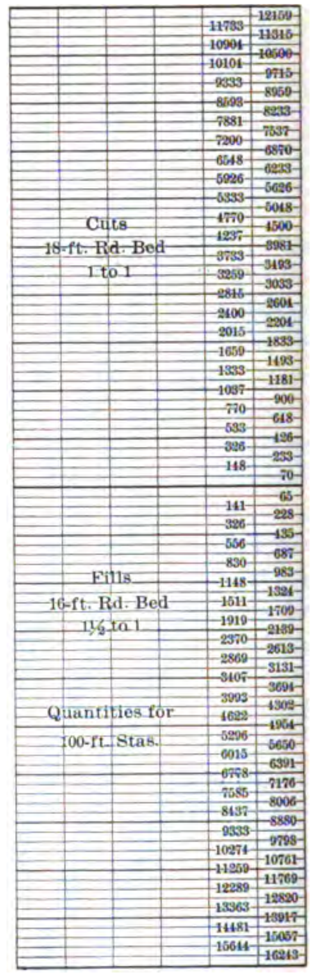
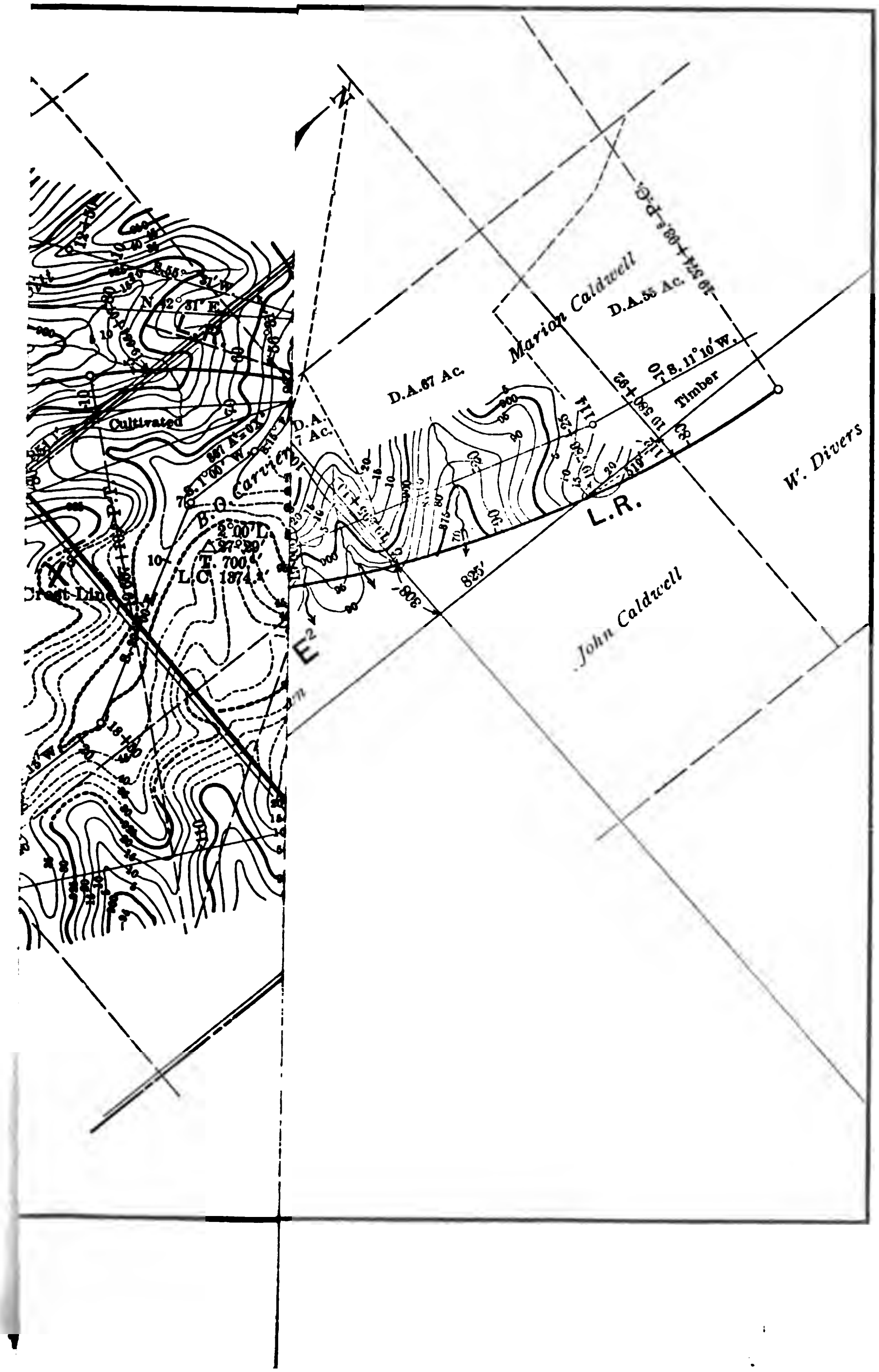
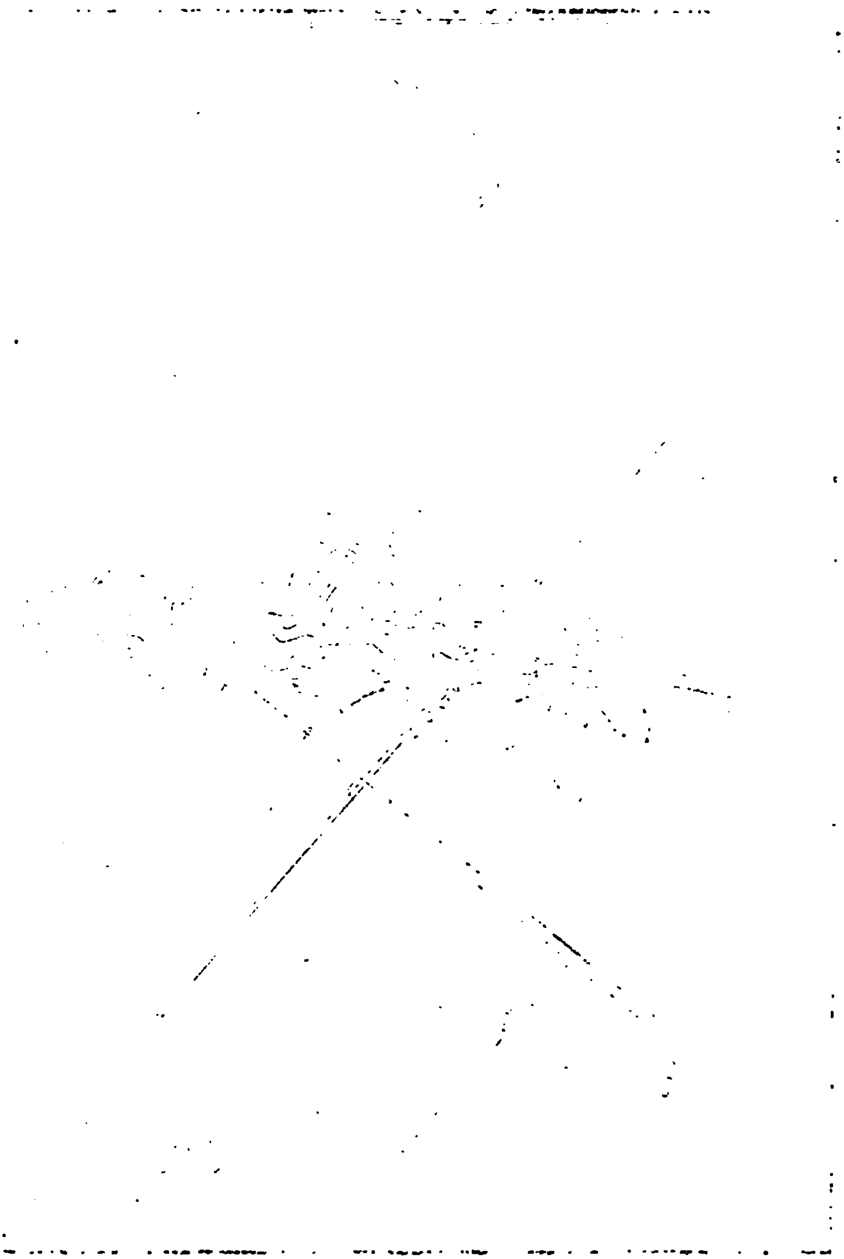


FIG. 12.

From time to time the Principal Assistant Engineer visited each locating party in the field, and thus kept in touch with the work and results of each. At the same time, the 5 000-ft. map and profile, etc., kept the record at headquarters complete. All projections adopted for location were examined and approved by the Principal Assistant Engineer, and, after approval, no deviation was permitted without authorization. By this means a detailed study of the line was possible; much more so than when viewed only on the ground by those superior to the Locating Engineer.


Marian Caldwell
D.A.55 Ac.
D.A.67 Ac.
S. 11° 10' W.
Timber
W. Divers
L.R.
John Caldwell
Cultivated
Crest Line
825'
E²



On the particular line in question, it was decided, by the time the different preliminary lines were nearly connected, that a 0.5% grade was possible for the whole length of the line, and instructions were given to make this the ruling grade on the final location.

Instructions were given the locating engineers to spend all the time necessary on investigations, to be sure they had the best line through the country traversed before putting in the location, on the ground. The writer recalls one stretch of line, about 16 miles in length, where he spent nearly three weeks, running over 80 miles of preliminaries, besides the original preliminary and projected location, before the final line was decided on. A second projected location saved a mile of distance over the first, besides eliminating much curvature and rise and fall, and, but for the very positive instructions received to exhaust every possibility, and the receipt, about this time, of a letter from the Principal Assistant Engineer, who knew the difficult nature of the country, reiterating his caution, this line would have been run in. Other lines were run, the final location effecting a saving of more than $30 000 in estimated cost of construction, and eliminating many degrees of curvature and more rise and fall.

It seems hardly possible, in view of this, which is only one case out of thousands, that anyone contemplating the construction of a railroad should hesitate to spend sufficient money on surveys, but all engineers of any extended experience know how difficult it often is to get either sufficient time or money to do this work thoroughly; and, as a result, how very much more the cost of the needless construction is likely to be than that of the surveys. Still, the writer believes it is often the fault of engineers in charge of work that this is so. Men now-a-days investing their money in any project of merit are as a rule level-headed business men who would be willing to furnish all the money necessary for proper surveys, if the matter were presented to them in the proper light.

As the final located line was run in, it was inked in on the 400-ft. map, radii of curves were drawn, stations of P. C. and P. T. marked, and calculated courses of tangents from observations of Polaris, length of tangents, the degree of curve, central angle, and the length of semi-tangents noted at each curve; drainage areas, as definitely determined by the topographer, were dotted in, and areas noted; as were also property lines and owners' names, thus making the map a complete record (see Plate XLIX).

In Indian Territory a right-of-way map on a scale of 2 000 ft. to 1 in. was made showing the alignment, station of P. C. and P. T. of curves, central angle, degree and length; also length and calculated course of tangents, all property lines and plusses to same, and property owners' names, where land had been allotted; ties to all section or quarter-section corners nearest the line, and notes in pencil where extra width for large cuts or fills might be necessary, the final width desired being added at headquarters. This map was required in Indian Territory only, to meet the Government requirements, the right of way being obtained by filing such a map with the Secretary of the Interior.

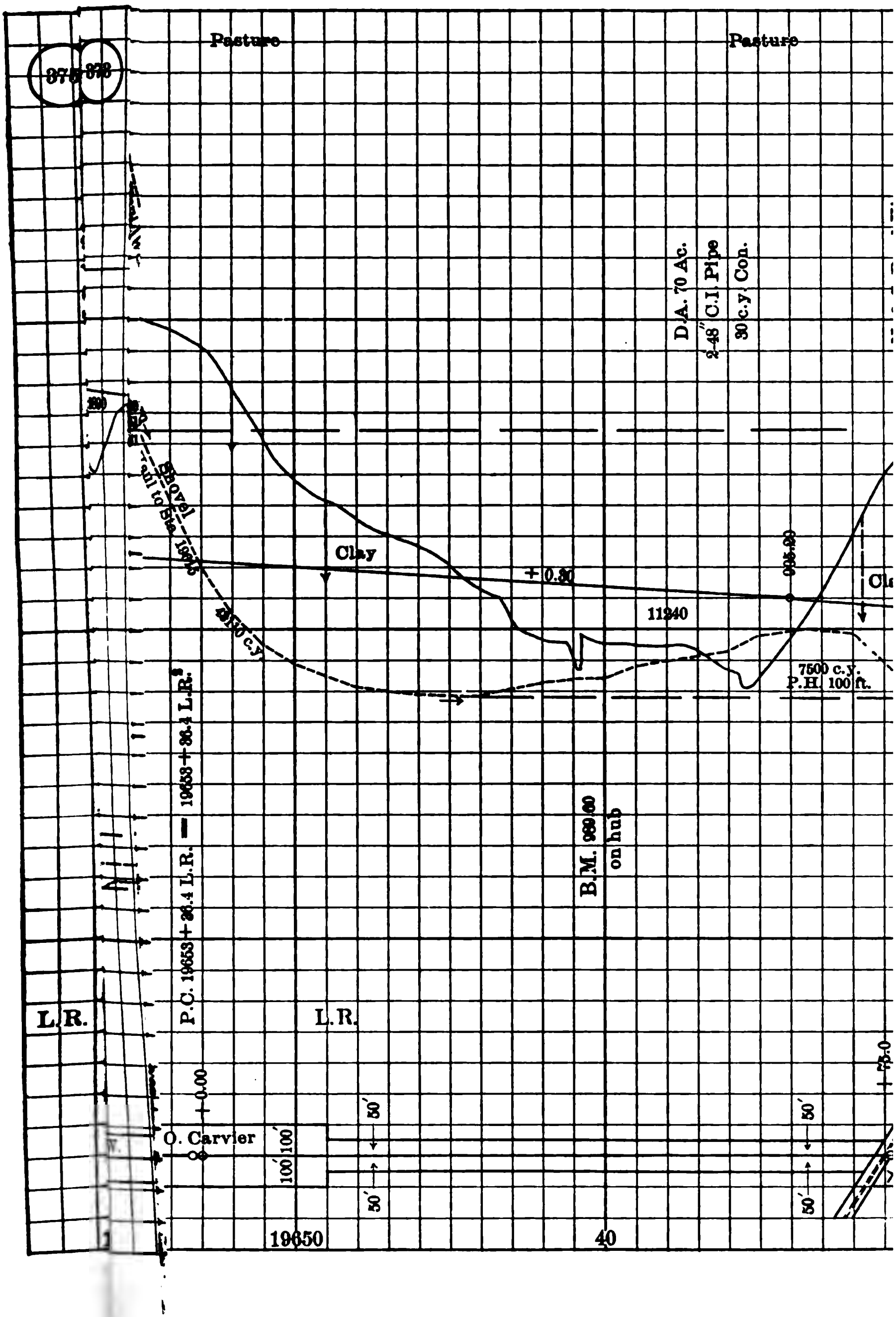
As the final profile and the ravine sections were platted, they were taken into the field by the Locating Engineer, and all bridge openings and culverts carefully fixed there. The profile as platted was inked in, but the grade line was left in pencil. As soon as the openings were fixed, and the soundings noted at the bridge sites and cuts, the estimate of quantities and the cost of each mile were made up by the draftsman and checked by other members of the party; this was then all carefully inked in, and a tracing made. Profiles of final locations were made in 25-mile sections. A portion of such a profile is shown in Plate L. This profile and the 5 000-ft. map contain all the information necessary to enable a contractor to bid intelligently on the work, and as all this work was kept up together, it was immediately available on the completion of the surveys.

The tracing of the profile of the final location was sent in to headquarters as soon as a 25-mile section was completed, together with the right-of-way map and ravine sections covering the same ground.

On receiving notice from headquarters that the grade line, as shown on the tracing profile, had been approved, the grade line on the original was inked in, any changes that were ordered being made, and then this was ready for the Division Engineer having charge of the construction of that section. Reference to the "Instructions to Resident Engineers" on the Choctaw, Oklahoma and Gulf Railroad, by Messrs. Molitor and Beard, will show how the work of construction was co-ordinated with that on the location.

All maps and profiles were carefully lettered in ink on the outside at each end, the lettering running parallel with the axes of the rolls, showing just what they were. On the 400-ft. map, the title might be something as follows:

PLATE L.
PAPERS, AM. SOC. C. E.
DECEMBER, 1904.
LAVIS ON
METHODS OF RAILROAD LOCAT



New York — Boston Line,
Providence — New London Section,
John Smith, Locating Engineer.
Final Location Sta. 852 to 1748, Mile 56 to Mile 73

Preliminary Sta.	A.	934	to	1823
" "	M.	48	"	223
" "	N.	15	"	329
" "	O.	0	"	56
" "	B.B.	17	"	638

The maps accompanying this paper are reproductions of maps actually made in the field, and show more clearly than any written description the kind of work accomplished.

The following is a statement showing in detail the cost of surveys conducted practically in accordance with the practice outlined in this paper. The length of the final located line in this instance was 179 miles, and the work was divided between four parties. The country was similar to that described by the writer, that is, long rolling country, rather badly broken up, the line running across the drainage, necessitating the exploration of a wide range of country on either side of the proposed route. The average amount of grading per mile was about 100 000 cu. yd., maximum grade, 0.5%, maximum curve, 2°; there was 19% of the line on curve. The writer is especially indebted to Mr. Beard for this information and notes on the same, as well as for much valuable assistance in the preparation of this paper.

Field preliminary expense for 563 miles..........	$14 628.97
" " " per mile..............	25.98
" Location " for 179 miles..........	12 597.92
" " " per mile..............	70.38
Locating Party No. 1: Expense on preliminary, incident to above location and including preliminary and location of 9 miles.................	2 478.02
Office expense charged to above..................	6 446.08
Total cost of preliminary and location 188 miles.....	$36 150.99
Total cost per mile.......................	$192.30

Preliminary Lines.

	Party No. 1. July 5th to October 1st. 87 days.	Party No. 2. July 23d to October 20th. 90 days	Party No. 3. August 1st to November 19th. 111 days.	Party No. 4. September 21st to October 21st. 30 days.
Miles run and topography taken	145.8	166.3	164.1	28.2
Miles run, no topography taken	39.3	....	16.0	3.6
Total miles preliminary run	185.1	166.3	180.1	31.8
Total number payroll days	1 380	1 323	2 033	635
Average daily number of men	15.9	14.7	18.3	21.2
Average miles per day per party	2.12	1.85	1.62	1.06
Total cost of subsistence	$513.18	$646.42	$763.53	$371.47
Average daily cost, subsistence per man	$0.37	$0.49	$0.38	$0.58
Relative cost percentage to lowest man subsistence	100	133	103	157
Total payroll cost (except teams)	$2 502.55	$2 683.22	$3 381.56	$1 055.55
Average daily pay per man	$1.81	$2.03	$1.66	$1.66
Total cost for teams	$522.00	$560.23	$768.55	$386.15
Daily " " "	$6.00	$6.22	$6.92	$12.87
Contingencies	$88.48	$112.95	$91.84	$125.78
Total cost of party	$3 629.96	$4 002.82	$5 057.96	$1 938.23
Daily " " "	$41.72	$44.48	$45.57	$64.61
" " per man	$2.63	$3.03	$2.49	$3.05
Cost per mile	$19.61	$24.07	$28.08	$60.95
Relative percentage to lowest man per mile	100	123	143	311

Located Lines.

	Party No. 1. 65 days.	Party No. 2. 37 days.	Party No. 3. 8 days.	Parties Nos. 2 and 3 Combined. 48 days.	Party No. 4. 66 days.
Miles located	56.0	37.8	7.6	42.6	39.2
Total number payroll days	1 400	709	151	1 498	1 283
Average daily number of men	21.5	19.0	19.0	31.2	19.4
Average miles per day per party	0.86	1.02	0.95	0.89	0.59
Total cost subsistence	$515.55	$273.07	$59.50	$599.20	$574.65
Average daily cost subsistence	$0.37	$0.39	$0.39	$0.40	$0.45
Total payroll (except teams)	$2 410.10	$1 143.11	$242.70	$2 562.74	$2 049.25
Average daily pay per man	$1.72	$1.61	$1.61	$1.71	$1.60
Total cost for teams	$434.74	$212.71	$43.10	$496.00	$445.85
Daily " " "	$6.69	$5.75	$5.39	$10.33	$6.76
Contingencies	$148.36	$46.76	$15.70	$196.00	$138.84
Total cost of party	$3 508.75	$1 675.65	$361.00	$3 898.94	$3 208.59
Daily cost of party	$53.90	$45.22	$45.12	$80.29	$48.54
Daily cost per man	$2.50	$2.36	$2.39	$2.57	$2.50
Cost per mile	$62.57	$44.33	$47.50	$90.47	$81.72
Relative percentage to lowest man per mile	141	100	107	204	184

There are various things to be taken into consideration in judging the fluctuations in the cost of these surveys. The preliminary location by Party No. 1 was over a severe country and embraced the heaviest work on the whole line; at the same time, much difficulty was experienced in getting a grade between certain points on the line located by Party No. 3. Party No. 2 had the lightest country.

There is charged to the expense of Party No. 4, the cost of moving a long distance from other work to this line, which amount, together with the short time they were engaged on preliminary, abnormally increased the cost of their work; at the same time, it is evident that this was decidedly the most expensive party on the work, their work per unit of cost, costing more.

For instance, their subsistence was 57% more than that of Party No. 1, and the team hire more than double that of the other parties; while the actual number of men in the field was relatively the same. It is probable that the cost of the work done by this party was really about 60% more than the others instead of 200% as shown by the cost per mile.

On location, Party No. 1 carried a very heavy and expensive sounding party, consisting of a man in charge, four or five laborers and a team; the nature of this work was such that it was much more expensive than that conducted by any of the other parties.

After completing the location, Party No. 1 was engaged in running other preliminaries and locating a short branch, the cost of this work not being distributed, but included in the total cost of the survey, the amount being $2 478.02.

On location, Parties Nos. 2 and 3 were combined after each had run in a short distance separately; this was necessitated by the approaching cold weather and the desire to complete the location at the earliest possible moment; the result shows it to have been an uneconomical proposition as far as cost per mile is concerned; but both of these parties had much additional preliminary work to perform as they proceeded with the location.

What has been noted of Party No. 4 on preliminary is true on location, though its cost is somewhat burdened by the charges incident to moving the party elsewhere, and the fact of its happening about Christmas, when many men were given vacations with pay. This Christmas expense was encountered to a somewhat less extent by every one of the parties, and tended to increase the total cost, but taken as to Parties Nos. 1, 2 and 3, the statement is a fair average of what a thorough survey under like conditions will cost.

Besides the organization as noted on page 871, which was practically the same as that engaged on these surveys, with the exception that there was only one topographer, there was the expense of an expert, at $150 per month and his expenses, engaged in an examination of the country adjacent to the line, for the purpose of determining the quantity of sand and stone available for construction purposes. The amount of this expense is $545.84.

In all this work it was considered absolutely necessary that all parts of it should be kept up together. and, with the large party available, it was found feasible to assign the men so that any part of the work which lagged behind could be brought up to date; the weak point is, of course, with the leveler on preliminary, if he gets behind there is little to do but to wait until he catches up. It is necessary, therefore, that an especially good man should be selected for this position. Physical ability to hustle is absolutely necessary, and the rodman must expect to trot between stations and the instrumentman between set-ups, if they cannot keep up by walking.

The leveler in a party should be, not only accurate, but quick. As an instance of what can be accomplished: On one of the lines referred to, starting from the west, more than 100 miles of preliminary were run to the eastern end of the line in 20 working days (not including Sundays and moving camp). On one day, the leveler covered 8 miles. On returning and making the final location, when every care was taken to have the levels as accurate as possible, equalization of sights being insisted on, and there being ample time for the leveler to do the work properly, no variation from any bench-mark was found greater than $\frac{5}{10}$ ft., the final check on the bench at the western end being about $\frac{3}{10}$ ft.

In making the preliminary location, or rather, the writer would prefer to say, in running the preliminary lines, he considers that the result to be obtained should be regarded more in the nature of making a topographical map of a strip of country through which the final location will pass, and through which runs a sufficiently accurate base line or lines, than in running a line which will be very close to the final location. There is only one place, in his opinion, to adjust the final location, and that is on a good topographical map.

This, of course, will not be misunderstood as relieving the Locating Engineer of the necessity of running these preliminary lines with judgment and a good idea of their relation to the located line. All the good judgment and "eye for country," relied on so much by some of the older locating engineers, are still as necessary as ever, but they must be supplemented by scientific methods and hard work.

The statement in regard to the final adjustment will possibly evoke some discussion from the many men who have saved thousands of dollars by slightly changing a curve in the field or otherwise after the final location is made, and the writer will admit, of course, that there is hardly a line located to-day, or likely to be, where every foot of it is exactly where it ought to be, but, in anything but the most minor changes, he believes that the fault will invariably be found in the fact that the original topographical map was not correct, or the projection not well made.

Provided the topography is generally correct, which it should be to be of any use at all, it is possible to project a line on it, which will be the best line the country affords, and, if the work is properly done, this line can be laid out on the ground. In adjusting the line to the topography, the line can be changed and a profile obtained fifteen times on the map while it is being changed once on the ground, and all the problems affected by the change studied.

The writer is well aware, of course, that the practice as outlined in this paper will necessarily be subject to many modifications to meet different conditions.

In conducting surveys in tropical countries or in other places where it is difficult to obtain experienced engineers, and then only at largely increased salaries (in tropical countries about two to three times as much as is noted in this paper), other methods become necessary, but the writer believes that the same ends should be striven for. In these cases, there is a great temptation to the engineer in charge of such work to shrink from the responsibility of insisting that he be given *carte blanche* by his employers in the matter of engaging such assistance as he may need and in the payment to them of adequate salaries.

There is much mountain country where transportation is extremely difficult, where everything possible must be done to lighten the equipment, and where a great deal of reconnaissance can be done in more or less detail with a light party, either by a separate party ahead of a larger one, or before a larger party is organized and put in the field.

Such a party can do much good work with a transit used as a level, or by using the stadia, in eliminating certain entirely impractical lines. In any event, under such conditions, both parties should be controlled by the same man, as what might have been regarded as impractical under one set of conditions may become entirely so under others; all this, however, is matter which will suggest itself to the experienced locator.

There are probably many old locating engineers, many who have done excellent work with much less equipment and fewer men, who will hold up their hands in horror against such an organization and equipment as is outlined here, but railroads themselves, as they exist to-day, are all the evidence necessary to prove that other methods than those of the past are necessary to meet changed conditions. Scientific methods must be applied to the conduct of location, as well as to the design of bridges, terminals, locomotives, etc.; in fact, on a proper location or otherwise the future of the railroad is almost entirely dependent.

In submitting this paper to the consideration of the Society, the writer does not wish to be understood as advocating any hard and fast rules for railroad location. No two lines are alike, topography is never the same, and nothing will take the place of experience, good judgment, and much hard work. He knows there are many good locating engineers who entertain different ideas, and he hopes they will submit them for consideration. He does firmly believe, however, that it will most certainly pay in the long run to obtain in every case, whatever the method may be, at least as much information as was obtained, on the work described, as shown by the maps and profiles accompanying this paper.

Vol. XXX. DECEMBER, 1904. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

MAXIMUM RATES OF RAINFALL AT BOSTON.

By Charles W. Sherman, M. Am. Soc. C. E.

To be Presented February 4th, 1905.

This paper is in effect a continuation of a paper entitled "Maximum Rates of Rainfall," by Desmond FitzGerald, Past-President, Am. Soc. C. E.,* in which he presented copies of the autographic records of the most important rain storms of high intensity recorded by the rain-gauge at Chestnut Hill Reservoir during the ten years, 1879–1888. The objects of the present paper are:

1.—To put on record in tabular form the important data relating to all storms in which the intensity of downpour was considerable, as recorded by the Chestnut Hill gauge from 1879 to 1904, inclusive;

2.—To present copies of the autographic records of rains of especial interest since 1888;

3.—To discuss briefly these records in comparison with those obtained elsewhere, with especial reference to the relation between the intensity and the duration of the downpour.

1.—Data of Maximum Rates of Rainfall at Chestnut Hill Reservoir, Boston, 1879–1904.

Table 1 contains the important data, except that very few storms having a duration of more than 10 hours are considered.

* *Transactions*, Am. Soc. C. E., Vol. XXI, p. 98.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

TABLE 1.—AMOUNT AND INTENSITY OF "DOWNPOUR" AS SHOWN BY RECORDING RAIN GAUGE AT CHESTNUT HILL RESERVOIR, BOSTON, MASS., 1879–1901.

(1) Date.	(2) Total Rainfall, in inches: By standard gauge at ground level.	(3) Total Rainfall, in inches: By recording gauge.	(4) Ratio of Column 3 to Column 2	(5) Amount of Downpour, in inches: As recorded.	(6) Amount of Downpour, in inches: Corrected.	(7) Duration of downpour, in minutes.	(8) Intensity of downpour, in inches per hour.
1879. June 29.......	1.41	1.07	0.759	0.86	1.18	45	1.51
July 16.......	....	0.72	.....	0.64	....	15	2.56
Aug. 16–19....	6.28	4.56	0.725	3.40	4.68	606	0.46
				0.35	0.48	15	1.93
Sept. 8.......	0.48	0.44	0.917	0.20	0.22	5	2.62
1880. July 20–21....	1.99	1.73	0.869	1.27	1.46	40	2.18
				1.00	1.15	18	3.83
1881. Jan. 10.......	2.20	2.20	1.000	2.20	2.20	800	0.17
				2.10	2.10	390	0.32
June 10–11....	3.83	3.50	0.914	3.50	3.83	2100	0.11
				2.50	2.73	480	0.34
July 21.......	0.48	0.36	0.750	0.35	0.47	12	2.33
Sept. 2–3......	1.79	1.74	0.972	0.76	0.78	35	1.34
1882. July 19.......	0.32	0.32	1.000	0.32	0.32	13	1.48
" "	0.48	0.43	0.896	0.40	0.45	18	1.49
Sept. 14.......	1.13	1.05	0.929	1 00	1.08	45	1.44
Sept. 22–23...	2.55	2.38	0.934	2.38	2.55	765	0.20
				1.50	1.61	165	0.58
1883. June 29.......	0.59	0.56	0.950	0.56	0.59	30	1.18
				0.40	0.42	8	3.16
1884. June 19.......	1.66	1.55	0.934	1.34	1.43	68	1.26
				1.55	1.66	240	0.42
July 18.......	1.19	1.14	0.958	0.67	0.70	5	8.40
				1.14	1.19	135	0.53
1885. June 28–29...	1.26	1.15	0.913	0.97	1.06	98	0.65
				1.15	1.26	470	0.16
June 29.......	1.17	1.05	0.897	0.93	1.04	68	0.92
				1.05	1.17	215	0.33
				0.60	0.67	15	2.68
July 29.......	0.81	0.76	0.938	0.76	0.81	125	0.39
				0.50	0.53	25	1.28
Aug. 1........	1.76	1.61	0.915	1.17	1.28	70	1.10
				1.61	1.76	365	0.29
				0.25	0.27	1	16.40*
1885. Oct. 2–3......	1.25	1.18	0.945	0.30	0.32	7	2.72
1886. July 15.......	1.74	1.59	0.914	1.59	1.74	450	.23
				0.30	0.33	8	2.46
				0.36	0.39	10	2.36
				0.37	0.40	15	1.62
1887. July 10.......	1.10	0.99	0.900	0.41	0.46	15	1.83
Aug. 20.......	0.33	0.30	0.909	0.30	0.33	8	2.48
Oct. 1	1.00	0.96	0.960	0.35	0.37	20	1.09
1888. May 18.......	1.32	1.22	0.924	1.22	1.32	420	0.19
				1.20	1.30	150	0.52
Aug. 12–13....	1.73	1.53	0.844	1.04	1.18	60	1.18
				0.96	1.09	45	1.45
Aug. 21–22....	3.44	3.23	0.939	3.02	3.22	200	0.97
				3.23	3.44	540	0.38
				1.50	1.60	60	1.60
				2.43	2.58	120	1.29
				3.06	3.26	180	1.09
Sept. 10......	1.35	1.30	0.963	1.24	1.29	190	0.41
Sept. 17–18....	1.94	1.78	0.918	1.17	1.27	120	0.64
				0.80	0.87	60	0.87

* Time interval estimated on diagram as not more than 1 minute. It is possible that float had been stuck and was released at this point, but diagram shows nothing to cast discredit on the record.

TABLE 1—(*Continued*).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Total Rainfall, in inches:			Amount of Downpour, in inches:			
Date.	By standard gauge at ground level.	By recording gauge.	Ratio of Column 3 to Column 2	As recorded.	Corrected.	Duration of downpour, in minutes.	Intensity of downpour, in inches per hour.
1889. May 20–21....	3.17	3.19	1.006	1.30	1.29	105	0.74
				0.55	0.55	25	1.31
				0.15	0.15	6	1.50
June 2........	1.59	1.55	0.975	0.30	0.31	9	2.06
June 5........	0.47	0.47	1.000	0.30	0.30	6	3.00
June 17.......	0.45	0.44	0.978	0.30	0.31	15	1.23
July 17.......	0.73	0.68	0.906	0.47	0.52	11	2.82
Aug. 1........	0.51	0.48	0.941	0.25	0.27	4	3.98
Aug. 14.......	1.60	1.51	0.944	0.31	0.33	12	1.64
Sept. 11......	1.35	1.04	0.770	0.20	0.26	5	3.12
1890. July 26.......	0.96	0.91	0.948	0.91	0.96	225	0.26
				0.83	0.88	60	0.88
				0.20	0.21	3	4.21
July 31.......	0.47	0.44	0.936	0.42	0.45	30	0.90
Aug. 6........	0.31	0.34	1.096	0.30	0.27	5	3.23
Aug. 19–20,...	0.75	0.72	0.960	0.40	0.42	11	2.27
1891. Sept. 5–7......	2.83	2.55	0.902	1.25	1.38	60	1.38
				0.50	0.55	15	2.22
Oct. 7–8	2.45	2.12	0.866	2.12	2.45	760	0.19
				1.90	2.19	270	0.49
				0.60	0.69	45	0.92
1892. June 17.......	1.18	1.07	0.907	1.07	1.18	110	0.64
				0.80	0.88	30	1.76
				0.52	0.57	15	2.29
July 3........	0.74	0.66	0.892	0.27	0.30	7	2.59
1892. July 25.......	0.44	0.34	0.773	0.32	0.41	15	1.65
Aug. 12.......	2.19	2.17	0.990	2.17	2.19	500	0.26
				1.30	1.31	30	2.62
				0.29	0.29	11	1.60
1893. May 27........	0.48	0.49	1.021	0.40	0.39	17	1.38
July 18.......	0.73	0.73	1.000	0.73	0.73	45	0.97
July 22.......	0.67	0.74	1.105	0.30	0.27	12	1.35
Aug. 6–7......	1.48	0.95	0.642	0.52	0.81	11	4.41
1894. July 21–22....	1.44	1.43	0.993	1.07	1.08	22	2.94
July 25.......	0.82	0.77	0.939	0.25	0.27	3	5.32
Aug. 20.......	1.40	1.42	1.014	1.40	1.38	280	0.30
				0.60	0.59	13	2.73
				1.00	0.99	35	1.69
Sept. 8........	0.34	0.34	1.000	0.24	0.27	7	2.12
1895. July 13.......	0.45	0.45	1.000	0.23	0.23	7	1.97
July 30.......	0.68	0.68	1.000	0.60	0.60	90	0.40
Aug. 7........	1.48	1.45	0.980	1.45	1.48	265	0.34
				0.35	0.36	8	2.68
Aug. 18.......	1.17	1.20	1.026	1.20	1.17	285	0.25
				0.80	0.78	30	1.56
				0.40	0.39	7	3.34
Sept. 11.......	0.75	0.75	1.000	0.70	0.70	17	2.47
Oct. 12–14....	7.55	....	....	....	....	2 325	0.19
Oct. 31–Nov. 1	2.86	2.43	0.850	1.50	1.76	180	0.59
Nov. 14–15.....	1.86	1.60	0.860	0.40	0.47	25	1.12
				0.25	0.29	9	1.94
1896. Sept. 5–6......	1.67	1.63	0.976	0.20	0.21	6	2.05
1897. June 15.......	0.32	0.32	1.000	0.30	0.30	9	2.00
July 28	1.74	1.57	0.902	0.88	0.98	50	1.17
				0.72	0.80	28	1.71
				0.62	0.69	20	2.06
				0.40	0.44	10	2.66
Aug. 4........	0.38	0.35	0.920	0.30	0.33	15	1.30
				0.16	0.17	4	2.61

TABLE 1—(*Continued*).

(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)
		Total Rainfall, in inches:			Amount of Downpour, in inches:			
Date.		By standard gauge at ground level.	By recording gauge.	Ratio of Column 3 to Column 2	As recorded.	Corrected.	Duration of downpour, in minutes.	Intensity of downpour, in inches per hour.
1897.	Aug. 22.......	0.62	0.66	1.064	0.57	0.54	20	1.61
	Sept. 16......	0.33	0.33	1.000	0.18	0.18	6	1.80
	Sept. 20......	0.98	1.00	1.021	0.56	0.55	15	2.19
1898.	July 4........	0.75	0.81	1.080	0.68	0.63	30	1.26
					0.59	0.55	15	2.19
	Oct. 21–22.....	1.49	1.25	0.839	0.15	0.18	3	3.57
1899.	July 8........	0.59	0.50	0.848	0.11	0.13	2	3.90
	July 26.......	0.89	0.82	0.921	0.11	0.12	6	1.95
	July 27.......	0.29	0.29	1.000	0.21	0.21	7	1.80
	Aug. 22.......	2.33	2.27	0.974	2.27	2.33	85	1.65
					1.65	1.69	22	4.62
					0.17	0.17	15	0.70
					0.41	0.42	15	1.68
					2.00	2.05	60	2.05
	Sept. 20......	3.34	3.18	0.952	3.18	3.34	460	0.44
					2.20	2.31	130	1.07
	Oct. 18........	0.61	0.54	0.885	0.30	0.34	25	0.81
1900.	Jan. 25........	0.57	0.55	0.965	0.13	0.13	8	1.01
	Aug. 15.......	0.39	0.40	1.026	0.25	0.24	15	0.96
	Aug. 16.......	0.72	0.67	0.930	0.10	0.11	3	2.15
	Sept. 17.......	2.02	1.80	0.891	1.30	1.46	180	0.49
					1.10	1.23	120	0.62
	Nov. 9........	0.49	0.47	0.959	0.30	0.31	8	2.35
1901.	July 29.......	2.10	1.90	0.905	1.90	2.10	330	0.38
					0.70	0.77	60	0.77
	Aug. 7........	1.45	1.43	0.986	0.35	0.35	10	2.13
	Aug. 25.......	2.10	2.20	1.047	1.95	1.86	65	1.72
1904.	Sept. 14–15....	3.84	3.78	0.984	3.56	3.62	510	0.43

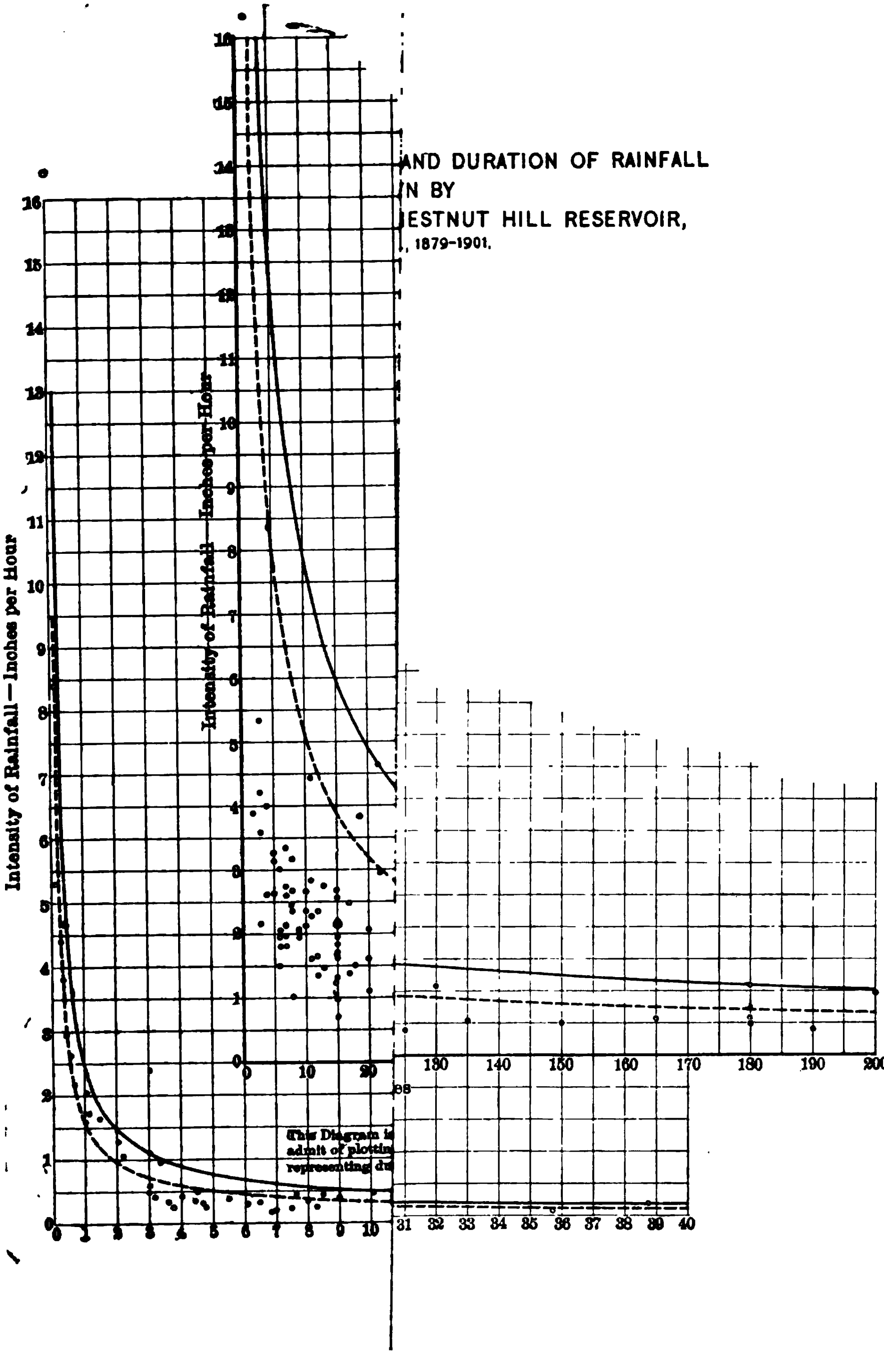
2.—Copies of Records.

The character of the records and the usual characteristics of the rain storms of high intensity are shown so well by the plates accompanying Mr. FitzGerald's paper that no useful object would be served by the presentation of other records, except in the case of storms of more than ordinary interest. Fig. 1 contains the records of three such storms.

The record for July 21st and 22d, 1894, showed a downpour amounting to 1.08 in. in 22 min., when corrected for elevation of the gauge, or a rate of 2.94 in. per hr.; and this rate was maintained with almost absolute uniformity for the whole 22 min.

The rain of August 22d, 1899, is one of the most interesting ever recorded by this gauge, in that it shows a precipitation in excess of 2

PLATE LI.
PAPERS, AM. SOC. C. E.
DECEMBER, 1904.
SHERMAN ON
MAXIMUM RATES OF RAINFALL.



in. in a single hour (to be precise, 2.05 in. in 60 min.). It also shows the unusual intensities of 1.65 in. per hr. for 85 min. and 4.62 in. per hr. for 22 min. As far as the writer is aware, but one other storm having a greater intensity than 2 in. per hr. for 60 min. has ever been registered by a recording rain-gauge in the eastern part of the United States, that of August 3d, 1898, at Philadelphia.*

The rain of September 20th, 1899, is of interest principally on account of the long time during which a comparatively high intensity was maintained, 1.07 in. per hr. for 2 hr. 10 min.

COPY OF RECORDS MADE BY RECORDING RAIN GAUGE.
OF CHESTNUT HILL RESERVOIR, BOSTON, MASS.

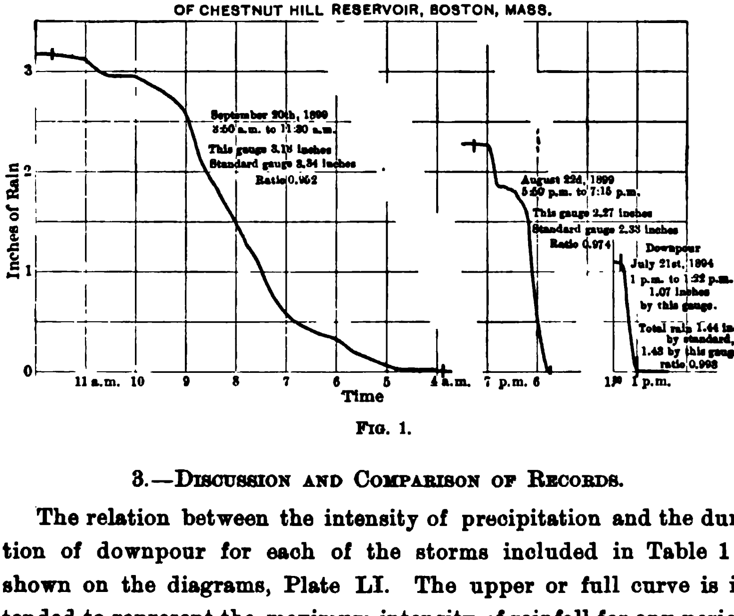


Fig. 1.

3.—Discussion and Comparison of Records.

The relation between the intensity of precipitation and the duration of downpour for each of the storms included in Table 1 is shown on the diagrams, Plate LI. The upper or full curve is intended to represent the maximum intensity of rainfall for any period, as far as it may be determined from these records. Its equation is $i = \frac{38.64}{t^{0.687}}$, in which i represents intensity of precipitation, in inches per hour; and t its duration, in minutes. It will be noted that none

* This most interesting storm, reported by Mr. A. J. Henry, of the U. S. Weather Bureau, in the *Journal* of the Western Society of Engineers for April, 1899, had intensities as follows:

7.20 in. per hr. for 5 min.
5.12 " " " " 15 "
4.02 " " " " 40 "
3.78 " " " " 70 "
2.96 " " " " 110 "

of the observed points falls beyond this curve. The lower or broken curve is intended to represent the greatest intensity of precipitation for any period which it would ordinarily be necessary to consider in engineering design, storms of greater intensity being of rare occurrence. It is represented by the equation $i = \frac{25.12}{t^{0.687}}$.

E. S. Dorr, M. Am. Soc. C. E., studying the records of this rain gauge, when the series included only 14 years, concluded that the expression $i = \frac{150}{t + 30}$ included all rainfalls which it would be necessary to consider in designing combined sewers for the City of Boston.* For periods greater than 20 min., this curve differs but slightly from the lower curve proposed in Plate LI, as may be seen by reference to Plate LII.

Comparison with other Records.—A. N. Talbot, M. Am. Soc. C. E., has noted that the maximum rates of rainfall seem to be very uniform in different parts of the United States (east of the Rocky Mountains), and has deduced a maximum and an ordinary curve as applicable to the whole country.† It is of interest, therefore, to compare with these records and curves those obtained from other observations. Such a comparison is shown on Plate LII, on which are plotted the following lines:

A. "Maximum" curve for Boston, proposed in this paper;

$$i = \frac{38.64}{t^{0.687}}.$$

B. "Ordinary" curve for Boston, proposed in this paper;

$$i = \frac{25.12}{t^{0.687}}.$$

C. "Ordinary" curve for Boston, proposed by E. S. Dorr in 1892;

$$i = \frac{150}{t + 30}.$$

D. Line of maximum rainfalls at Chicago, as determined by Edwin Duryea, Jr., M. Am. Soc. C. E.‡

E. Line of maximum intensity of rainfall at Washington for the 16 years, 1881–1896, as reported by Mr. Alfred J. Henry.§

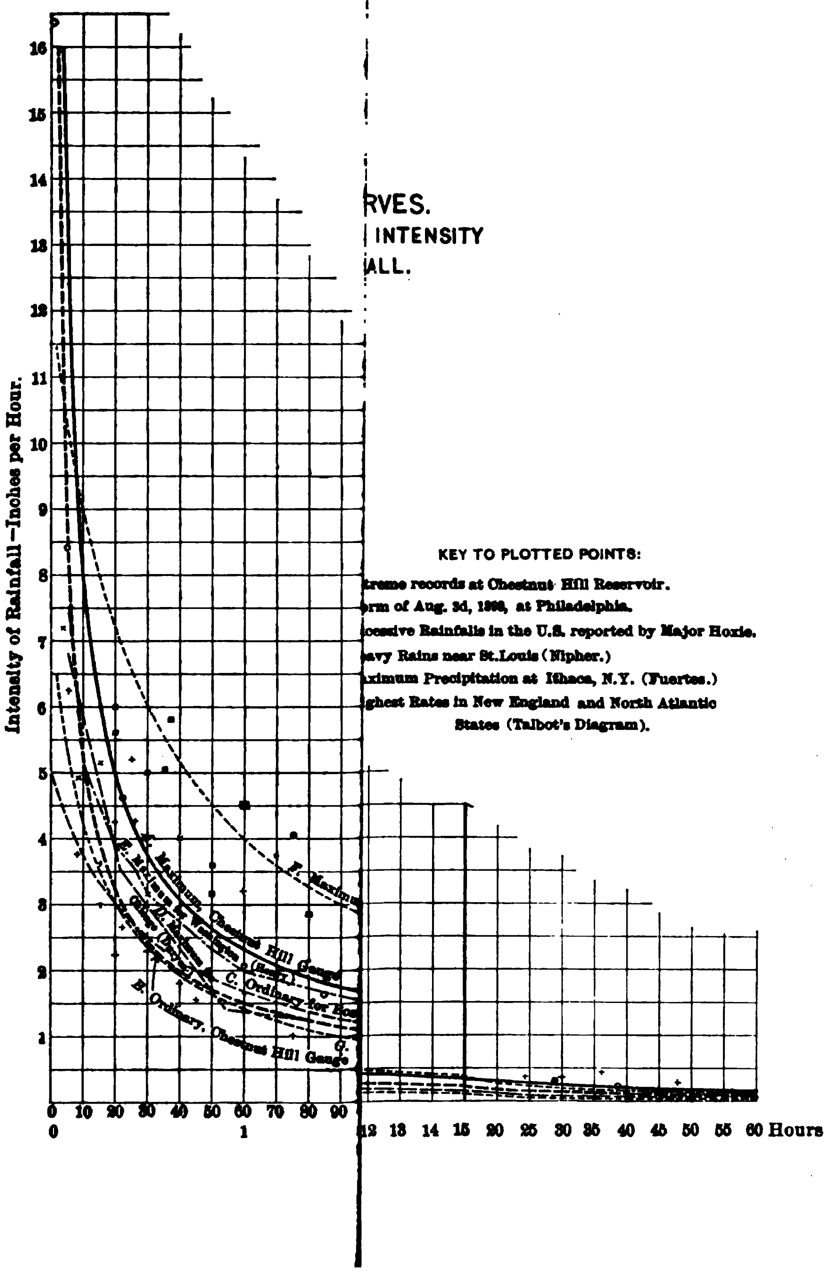
* "Report of Street Department, Boston," 1892, p. 117.

† "Rates of Maximum Rainfall," *The Technograph*, 1891-92. Abstracted in *Engineering News*, July 21st, 1892, p. 67.

‡ "Tables of Excessive Precipitations of Rain at Chicago, Ill., from 1889 to 1897, Inclusive." *Journal* of the Western Society of Engineers, February, 1899.

§ "Excessive Precipitation in the United States." *Monthly Weather Review*, January, 1897. (Abstract in *Engineering News*, June 24th, 1897, p. 386.)

PLATE LII.
PAPERS, AM. SOC. C. E.
DECEMBER, 1904.
SHERMAN ON
MAXIMUM RATES OF RAINFALL.



F. "Maximum" curve for eastern United States, proposed by A. N. Talbot, M. Am. Soc. C. E.;*

$$i = \frac{360}{t + 30}.$$

G. "Ordinary" curve proposed by Professor Talbot;

$$i = \frac{105}{t + 15}.$$

In addition to these curves, points have been plotted on Plate LII to show excessive rates of rainfall noted at other stations in the United States, as well as the extreme rates recorded at Chestnut Hill Reservoir. These points are:

a. Extreme records at Chestnut Hill Reservoir. (Marked by circles.)

b. Storm of August 3d, 1898, at Philadelphia.† (Marked by crosses.)

c. All the excessive rainfalls reported by Major R. L. Hoxie, M. Am. Soc. C. E., which occurred in the United States.‡ This includes the great rain of October 3d and 4th, 1869, reported by the late James B. Francis, Past-President, Am. Soc. C. E.;§ also two storms reported in Trautwine's "Civil Engineer's Pocket Book" as follows:

"In July, 1842, 6 inches fell in 2 hours. . . . During a tremendous rain at Norristown, Pa., in 1865, the writer saw evidence that at least 9 inches fell in 5 hours." (Points marked by plusses.)

d. Heavy rains near St. Louis, reported by Francis E. Nipher in a letter to the *American Engineer*, May 8th, 1885. (Marked by crosses within circles.)

e. The maximum precipitation registered by the recording rain gauge at Ithaca, N. Y., as reported by James H. Fuertes, M. Am. Soc. C. E. ‖ (Marked by a triangle.)

f. The highest points shown on Professor Talbot's diagram for the New England and North Atlantic States, in the paper previously quoted. (Marked by rectangles.)

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* *The Technograph*, 1891–1892; *Engineering News*, July 21st, 1892.

† A. J. Henry in *Journal* of the Western Society of Engineers, April, 1899.

‡ "Excessive Rainfalls Considered with Especial Reference to their Occurrence in Populous Districts." *Transactions*, Am. Soc. C. E., Vol. XXV, p. 70.

§ *Transactions*, Am. Soc. C. E., Vol. VII, p. 224.

‖ "Rates of Precipitation in Rain Storms at Ithaca, N. Y." *Engineering News*, September 20th, 1894, p. 226

In examining Plate LI it is at once seen that, with the exception of Professor Talbot's "maximum" curve, the "maximum" curve obtained from the Chestnut Hill records is the highest of any; and that the "ordinary" curve derived in this paper does not differ widely from Professor Talbot's "ordinary" curve, nor from any of the others shown.

With reference to the points lying above the Chestnut Hill "maximum" curve, it should be remembered that the only ones obtained by a recording rain gauge are those representing the Philadelphia storm of August, 1898. All the others, therefore, are open to question to a greater or less degree. The Philadelphia record, however, at once lends credence to the others as being at least approximately correct. It appears, then, that Professor Talbot's "maximum" curve must probably be accepted as representing an intensity of rainfall that should be expected perhaps once in a century; except that this curve seems to give results which are too small for durations of less than 5 min. or more than about 15 hr. It appears, also, that, although there is nothing in the Chestnut Hill records to support such a conclusion, such extreme rainfalls must be expected at Boston, since there have been a number of sufficiently well authenticated storms of this character at no great distance from Boston, as well as the autographically recorded storm at Philadelphia in 1898.

Finally, it would appear that, for ordinary engineering design, such a rainfall as would be shown by either of the three "ordinary" curves—Mr. Dorr's, Professor Talbot's, or that proposed in this paper—would be as heavy as there would usually be necessity for considering. The writer believes the last curve to be better supported by the Chestnut Hill records. In extreme cases it may be necessary to consider rainfalls as heavy as those shown by the "maximum" curve herein proposed. Such rainfalls may perhaps be expected as often as once in 8 or 10 years. Finally, such extreme rainfalls as would be shown by Professor Talbot's "maximum" curve (between the limits of 5 min. and 15 hr., beyond which this curve gives results which are too low) must be expected to occur at long intervals, perhaps about once in a century.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

A RATIONAL FORM OF STIFFENED SUSPENSION BRIDGE.

Discussion.*

By C. C. Schneider, M. Am. Soc. C. E.

C. C. Schneider, M. Am. Soc. C. E. (by letter).—Suspension bridges with eye-bar chains in which the stiffening truss is attached to the chain have been built before, though on a smaller scale; in fact, the eye-bar chain was used before the wire cable in suspension bridges. Mr. Schneider.

The distinguishing features in the author's design are:

The shape of the stiffening truss and the tower;

The shape of the stiffening truss varying in height so as to conform with the theoretical requirements.

The latter is desirable for the greater stiffness it produces, as well as its economy.

The tower, consisting of vertical posts with the chain fastened to the top and free to rock on a pin-bearing below, the writer considers an improvement on the braced tower with the chain or cable fastened to a saddle moving on rollers.

The author's design of a stiffened suspension bridge compares favorably, for rigidity, with other types of bridges, and is destined to be a rival of the cantilever for long-span railroad bridges.

Rigidity is a desirable quality in any permanent structure, and more particularly in a structure of such magnitude, which should be built to last for generations.

*Continued from November, 1904, *Proceedings*. See August, 1904, *Proceedings* for paper on this subject by Gustav Lindenthal, M. Am. Soc. C. E.

Mr. Schneider. The late George S. Morison, Past-President, Am. Soc. C. E., expressed himself in 1901 on the question of stiffness as follows:*

> "Bridge specifications, many of which are drawn with great care, generally have the defect of specifying strength and not providing for stiffness. Under such specifications two structures can be accepted as equally good, though, under a load, one of them will have twice the distortion that the other will have. For immediate safety, strength is all that is needed; for long life of structure, stiffness is at least equally important. This objection may be raised to some special forms of trusses, favored because of their economy of material, in which much greater variations occur in the strains of individual members than in differently designed trusses containing a little more metal."

Some of the discussors have endeavored to make a comparison of cost between the wire-cable supension bridge with an auxiliary stiffening truss, and the author's design. A fair comparison is only possible if the designs of the two types of bridges are worked out under the same specifications and requirements, with the same unit strains for the floor system and stiffening trusses in each case.

The relative proportion of permissible unit strains on wire cable and eye-bar chain has been established, in the Brooklyn and Williamsburgh Bridges, between cable and anchorage chain, which has never been questioned as being good practice.

The anchorage chain in the Brooklyn Bridge, with iron eye-bars, has approximately 3¼ times the section of the wire cable; and the anchorage chain of Williamsburg Bridge, with steel eye-bars, 3 times the section of the wire cable. Considering the difference in strength between the iron and steel eye-bars, the proportion in both cases is nearly the same.

In accordance with this established practice, the relative unit strains for ordinary steel eye-bars and wire cable should be as 1 to 3, and, for nickel-steel eye-bars, with 50% greater strength than carbon-steel eye-bars, this proportion becomes 1 to 2. If, therefore, a working strain of 60 000 lb. per sq. in. is allowed on the wire cable, 30 000 lb. per sq. in. should be allowed on the chain of nickel-steel eye-bars.

Bids on the two designs should be asked at the same time, in order to have the same conditions as to prices of material, labor, etc. This, in the writer's opinion, is the only way to satisfy the public as to which of the systems is the more economical. If one design produces a stiffer bridge than the other at approximately the same cost, the stiffer bridge will be the cheaper in the end.

The writer, in examining the feasibility of the author's design, from a manufacturer's point of view, fails to find any difficult or unusual features, as far as the details of the structural steelwork are concerned. The details and connections can be designed so as to re-

* *Transactions*, Am. Soc. C. E., Vol. XLVI, pp. 39 and 40.

duce the same to ordinary simple bridgework, such as any modern bridge shop equipped for heavy work can manufacture. Mr. Schneider.

The members of the stiffening truss are of moderate sizes, and the maximum sectional area of the eye-bar chain is only slightly in excess of the maximum area of the tension chord of the Blackwell's Island Bridge.

From a manufacturer's standpoint, the work involved in the author's design of a stiffened suspension bridge is more desirable for a bridge shop than that of the Blackwell's Island Bridge, as it is more uniform and presents fewer complications in its details. Heavier and more complicated work has been manufactured, and more difficult problems of erection have been solved heretofore.

Some of the discussors have been questioning the propriety of the use of eye-bars for the cables. The eye-bar has been the principal distinguishing feature of American bridge building from the beginning of iron bridge construction. To condemn eye-bars as tension members in bridges would be a condemnation of the entire American practice of bridge building. Eye-bars as tension members have stood the test of time and have proved so satisfactory that they are now used exclusively in all railroad bridges of long span. To the writer's knowledge, no case is on record of a bridge failing on account of eye-bars being used as tension members.

Eye-bars, in connection with pin-connected trusses, have only proved unsatisfactory where they have been used improperly. In the early days of bridge building, when economy in weight of material was the first and almost only consideration of bridge designers, pin-connected trusses with eye-bars were used for very short spans, and consequently lacked the rigidity required in good railroad bridges. The writer has for years been an advocate of substantial riveted work for railroad bridges for small spans, and has been very persistent in his condemnation of the use of small eye-bars. Even at the present time eye-bars are abused. The writer, only recently, has seen pin-connected highway bridges of 30 to 40 ft. span built with 1½ by ½-in. eye-bars. Any eye-bar less than 3 by ¾ in. should be condemned, even in the lightest kind of highway bridges. If the required section in the tension member is too small for 3-in. eye-bars, riveted members should be used, irrespective of the length of the span. Pin-connected trusses with eye-bar tension members for single-track railroad bridges of more than 200 ft. span and for double-track bridges of more than 150 ft. span are at the present time considered good practice by all American bridge engineers. Eye-bars have been used successfully in all the large cantilever bridges built recently, and will be used in the Quebec Bridge, which, when completed, will be the longest span in the world. It is well known to those who have had experience in the manufacture of eye-bars that they are generally the cheapest members

Mr. Schneider. in a bridge. The cost of manufacture per pound decreases as the size and length of the bar increases. The larger and longer the bar, the cheaper the cost per pound. Eye-bars of 14 and 15 in. in width have been manufactured in large quantities, and a number of experimental bars, 16 by 2 in., have also been manufactured successfully. Either of the above-mentioned sizes could have been used in the author's design. Eye-bars are used in the anchorages of most of the large suspension bridges, such as the Brooklyn and Williamsburgh Bridges, and, as the anchorage is a continuation of the cable, there is no good reason why eye-bar chains should be condemned for one portion of the cable and used for the other.

The eye-bar chain has the advantage that the sectional area can be varied to accommodate the variations of the strains in the same, while it is necessary to give the wire cable the required maximum sectional area throughout its entire length.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
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PAPERS AND DISCUSSIONS.

PROBABLE WIND PRESSURE INVOLVED IN THE WRECK OF THE HIGH BRIDGE OVER THE MISSISSIPPI RIVER, ON SMITH AVENUE, ST. PAUL, MINN., AUGUST 20TH, 1904.

Discussion.*

BY MESSRS. THEODORE COOPER AND GEORGE E. GIFFORD.

THEODORE COOPER, M. AM. SOC. C. E. (by letter.)—Mr. Turner deserves the thanks of the Profession for his investigation of the wreck of the St. Paul bridge. His explanation of the failure appears to be the probable one. For many years the writer has watched the reports of tornadoes and their effects, and has yet to find a case of a properly designed and constructed bridge which has been destroyed by the wind. Mr. Cooper.

While many bridges, roofs and other structures have been overturned or destroyed by the wind, the writer has not found one case which indicated that the modern requirements for wind bracing have proved inefficient. In numerous cases, the wind has been made the cloak to cover the ignorance, inexperience, neglect or chicanery of the designer or builder. The wind has been very much maligned, and, in the writer's opinion, its power has been very much over-estimated.

* This discussion (of the paper by C. A. P. Turner, M. Am. Soc. C. E., printed in *Proceedings* for November, 1904), is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion.
Communications on this subject received prior to January 28th, 1905, will be published subsequently.

Mr. Cooper. While preparing the Erie Specifications, in 1879, the writer found a memorandum, by the late George S. Morison, Past-President, Am. Soc. C. E., formerly Principal Assistant Engineer, giving the sizes of lateral rods in such spans as were acting stiffly under the trains. Using this as a guide, and also finding that it corresponded very closely with the results of 30 lb. per sq. ft. of exposed surface, a lateral force per linear foot of span was adopted for "wind and vibration."

A few years ago it was brought to the writer's attention that, under modern train loads and speeds, this rule did not give sufficient rigidity, and in his later specifications it has been increased one-third.

This amount of lateral stiffness, therefore, is needed in bridges of ordinary lengths of span, regardless of the wind, and, as it has proved satisfactory under the wind forces also, there is no reason for changing it, except it be found desirable to give increased stiffness under moving loads.

All authorities agree that an absolutely steady wind is unknown; that the wind is a series of eddies and gusts; that the records of anemometers are only the measures of the highest gusts upon small areas and only represent an instantaneous and limited effect. It follows naturally that on larger areas the average pressure at any moment will be less than that recorded by the pressure gauge. At the Forth Bridge the pressure on a surface of 20 by 15 ft. was practically only about two-thirds of that shown on a gauge having an area of 1½ sq. ft. For larger spaces the reduction would probably be still more marked. The irregular waves of a field of grain acted upon by the wind make clear to an observer the effects of the gusts and why on large spaces the full effect is never possible at any one instant.

Since Smeaton's time a wind of 100 miles per hour has been rated as one that would uproot trees and move buildings. Such a wind, according to accepted formulas, would exert a maximum pressure of 40 lb. per sq. ft. on small areas. A pressure of 25 lb. per sq. ft. acting at the same moment on large areas would, in the writer's opinion, denude a district of trees and buildings, and it is very unlikely that a wind of 100 miles' velocity exerts a force greater than this on objects of ordinary size.

General Greeley, of the United States Signal Service, in 1890, after the Louisville, Ky., tornado, stated:

> "As bearing upon the strength of structures necessary to withstand tornadic winds, it is important to note that there have been very few cases recorded of wind velocities in the United States where the pressure of the wind, according to the latest investigations and accepted formulæ, exceeded 16 lb. to the square foot."

The severest effects of the wind occur in the paths of tornadoes. These, however, are very limited in their breadths, the high pressures or destructive effects being limited to a few hundred feet, and then the results are usually recognized as being due to the oscillation of the path of a single vortex or of a series of vortices following one another. It is probable that the destructive action of a single vortex does not exceed a breadth of about 60 ft., as has been frequently observed. Moreover, being a rotating force, it could not exert its pressure in any one direction for more than half its breadth. Mr. Cooper.

The estimated pressures or velocities to produce the greatest recorded results, such as lifting locomotives, breaking off the top of an obelisk, twisting iron bars, driving straws through pine boards, etc., amount to only about 150 lb. per sq. ft. or 200 miles per hour.

Julius Baier, M. Am. Soc. C. E., in his excellent paper on the last St. Louis tornado,* found evidences of pressures as high as 60 lb. over a length of 180 ft.

At the St. Charles Bridge, the late C. Shaler Smith, M. Am. Soc. C. E., reported a tornado which exerted a pressure of 52 lb. on small objects on the bridge and 84 lb. in the vicinity, and which did not injure the bridge, although its bracing was only proportioned for 30 lb. per sq. ft. of exposed surface. He also found, after following up the paths of several tornadoes, but one case where 60 ft. of width was not enough to cover the path within which the computed pressures exceeded 30 lb.

Many careful investigators of the effects of high winds and tornadoes have concluded that it is very improbable if they ever exert an effect of 30 lb. per sq. ft. over a space of 150 to 200 ft., at one time.

When one considers then the lateral force for which bridges have been designed and the limiting strains allowed for all bracing, one has a right to expect that any properly designed and constructed bridge will escape injurious effects from high winds or tornadoes. However strong the bridges may be braced, if they are not properly anchored down or stayed against being shoved off their seats—two very common faults—they may at some time fail.

Regardless of the very strong probability that a lateral force equal to the ordinary requirements of our specifications is never exerted by the highest winds on spans of more than 150 ft. and not oftener than once in a generation on shorter spans, the bracing due to such requirements could not be reduced without rendering these bridges inefficient against the vibrations of moving loads. But an era of very long spans has been entered, in which it is desirable and necessary to take a broader view of this subject.

To design a 2 000 to 3 000-ft. span, which the writer believes will not be an uncommon length for the next generation of bridge builders, and may even occur as a case for some of the present generation, under the same wind requirements as for spans of 500 ft. and less, would be very unscientific and wasteful.

* "Wind Pressure in the St. Louis Tornado with Special Reference to the Necessity of Wind Bracing for High Buildings," *Transactions*, Am. Soc. C. E., Vol. XXXVII, p. 221.

Mr. Cooper. The development of these longer spans and the limitation of the greatest possible span depend very largely upon the assumed wind force. The wind force, being practically a horizontal force, while the dead and live loads are vertical forces, any unnecessary material added for impossible wind forces is a detriment to the structure and a handicap against progress.

Sir Benjamin Baker gives the following resultant stresses per square inch, on the top and bottom members of the Forth Bridge for dead, live and wind forces, in tons.

	Dead load.	Live load.	Wind.	Total.
Top member..........	4.4	2	1.1	7.5
Bottom member.......	2.8	1.2	3.5	7.5

Bearing in mind that an important part of the dead load was due to the material added for the wind stresses, it will be seen that in the lower members the wind exerted a greater influence than the dead and live loads together.

The Board of Trade required the Forth Bridge to be built to resist a "wind force of 56 lb. per sq. ft., striking the whole or any part of the bridge at any angle upon an area equivalent to twice the plane surface of the front girders, with a reduction of 50% in case of tubes."

Fifty-six pounds per square foot of surface striking simultaneously a length of 1 700 ft., when the highest possible claim that could reasonably be made from the evidences of the worst known storms would not give this pressure over 200 ft.!

It may be unnecessary to say that this requirement was imposed upon the eminent engineers of this bridge, and was not the result of their own conclusions.

It might be possible that a wind force of 56 lb. per sq. ft., covering a lateral extension of 1 700 ft., through its oscillatory movements, could occur, but, from all the evidence of the character and action of such high winds, it is impossible to conceive of such a wind striking a simultaneous blow of this force over 100 ft. of lateral distance.

The absurd requirement forced upon the Forth Bridge should not be accepted as a precedent. It is time that a more rational requirement for long spans should be attempted.

For spans up to about 500 ft. the existing requirement of a fixed lateral force per foot of span should not be relaxed, as it is desirable to have this much rigidity against the action of moving loads.

For very long spans, where the exposed surfaces become much larger, the lateral force sufficient to give rigidity under moving loads may not be enough to provide for possible wind forces.

For all spans exceeding 500 ft. in length the writer would suggest the following wind forces as sufficient to cover all cases.

First.—A wind force of 50 lb. per sq. ft. acting at the same moment over a width of 60 ft., striking any part of the bridge at any angle within 30% above or below the horizontal;

Second.—Similarly, a wind force of 30 lb. over a width of 600 ft.; Mr. Cooper.

Third.—Similarly, a wind force of 15 lb. over a width of 2 000 ft.; the maximum stresses from either of these requirements to be used for proportioning each member.

As all these requirements are far beyond what recorded evidence would lead us to believe as probable; as their duration, should they occur, is for a very short time, acting on a mass of great inertia; and as their recurrence would only be at long intervals of time, engineers would be justified in using high unit strains for the combined dead, live and wind loads. Assuming that for very long spans two-thirds of the elastic limit of the material for dead and live loads combined is not exceeded, they could use safely $33\frac{1}{3}\%$ more, or eight-ninths of the elastic limit for the dead, live and wind strains combined, for the truss members.

Though the foregoing wind requirements, in the writer's opinion, are excessive, they are so much more reasonable than the usual ones specified for long-span bridge projects that it is desirable to have them discussed.

Their acceptance, after such modifications as may be developed by discussion, would vastly improve the possibilities of future long-span bridges.

GEORGE E. GIFFORD, M. AM. SOC. C. E.—The speaker believes that Mr. Gifford. the probable cause of the disaster which forms the subject of Mr. Turner's paper is correctly stated by the author, and in this also agrees with Mr. Cooper.

There is one exception which might be taken to the conclusions drawn by the author: His statement, or rather, intimation, that a riveted structure with stiff members throughout would have been preferable, does not seem to be borne out by the facts. It is not clear that the same thing would not have happened, whatever the type of structure, since the failure seems to have been caused by deficient bolting to supports, or lack of dead weight. The dead weight might have been greater, and probably would have been, had it been a riveted structure, but the author does not take this into account. Nor would it have been sufficient to hold the truss down, under the conditions stated, in the absence of proper bolting.

The speaker does not at this time intend to discuss the merits of riveted *versus* pin-connected trusses, but it is questionable whether a riveted truss is preferable, all things considered, for a highway span of 250 ft. It certainly cannot be fabricated in the shop at a sufficiently lower cost to offset the additional material required and extra cost of erection.

Vol. XXX. DECEMBER, 1904. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE STRUCTURAL DESIGN OF BUILDINGS.

Discussion.*

By Messrs. L. J. Johnson, H. P. Macdonald, E. P. Goodrich, M. S. Ketchum, G. H. Blakeley, John B. Clermont and Oscar Lowinson.

Mr. Johnson. L. J. Johnson, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Schneider has certainly done the profession a great service in publishing this paper. The writer begs to express his share of the gratitude for it. While so doing, he wishes to discuss one point which Mr. Schneider brings up —the weight of a crowd of people. He is not reconciled to such low figures as Mr. Schneider cites for this quantity. By his recommendations and his assertion, that "a unifoim load of 40 lb. per sq. ft. will scarcely ever be exceeded by a crowd of people," he may give the impression that 40 lb. per sq. ft. is a fair estimate of the actual weight of a closely packed crowd, an impression which will not be found to be substantiated by facts.

A few months ago the writer made some experiments on the weights of crowds of his students, and found that 156 lb. per sq. ft. was attainable without any attempt at selecting the men or crowding them to any painful degree of personal discomfort. Results nearly as high are reported by Mr. Stoney, by Professor Kernot, of Victoria, and by C. M. Spofford, Assoc. M. Am. Soc. C. E.†

This knowledge is often important in design. Allowance for it, of course, may be made in any way that may be approved by the judg-

* Continued from November, 1904, *Proceedings*. See September, 1904, *Proceedings* for paper on this subject by C. C. Schneider, M. Am. Soc. C. E.

† *Engineering News*, Vol. LI, pp. 360 and 426.

FIG. 1.—CROWD WEIGHING 1 505.8 LB., TOTAL; OR 41.6 LB. PER SQ. FT.

FIG. 2.—SAME CROWD AS IN FIG. 1, BUT LESS SCATTERED.

FIG. 3.—CROWD WEIGHING 3 013.4 LB., TOTAL; OR 83.8 LB. PER SQ. FT.

ment of the designer, but, in any event, it should be understood clearly that 150 lb. per sq. ft. may be reached, or even exceeded, under a crowd of people at points subject to special congestion. Mr. Johnson.

For example, students among those who formed the crowd above mentioned expressed the conviction that the throngs leaving the university football field after large games are at one place compressed quite as tightly as were the students in this test. This special congestion occurs upon a drawbridge the width of which between railings is considerably narrower than the street which leads to it. The writer has vivid recollections of being in crowds compacted by the bridges at the Columbian Exposition on Chicago Day, where the density of the crowd must have been quite as high as that of the crowd of students when weighing 156 lb. per sq. ft.

While 150 lb. per sq. ft. may be reached only rarely, 80 or 100 lb. per sq. ft. must be realized far more often than is commonly supposed. A crowd of 80 lb. per sq. ft. can easily make way so as not to afford serious obstruction to the progress of a person who wishes to go through it, and a little persistence will enable a person to make his way through a willing crowd weighing 120 lb. per sq. ft. The details of the experiments on which these assertions are based, and a collection of citations from American and foreign authorities will be found in *Engineering News.**

It has occurred to the writer that photographs, showing just what degree of congestion is indicated by loads of about 40, 80, 100 and 150 lbs. per sq. ft., would be of interest at this point. Consequently, a number of volunteers from among the writer's students were weighed and caused to stand in a box made for the purpose. This box was 6 ft. square, in the clear, inside measurement, and with vertical walls, 5 ft. 9 in. high, and without a top. The men filed into the box, and photographs were taken as the weights reached the requisite totals. The camera was at a window some 20 ft. above the top of the box, and the men were asked to look up, so as to be more readily identified and counted, as a check upon the record. The results are shown on Plates LIII and LIV.

Figs. 1 and 2, Plate LIII, show the same group of students. In Fig. 1 they are distributed over the available area, and in Fig. 2 they are assembled along one side of it. These ten men aggregated in weight 1 505.8 lb., which, on the 36 sq. ft., made a load of 41.8 lb. per sq. ft.

Fig. 3, Plate LIII, shows the same men and ten additional men, bringing the total up to 3 013.4 lb., and unit load to 83.7 lb. per sq. ft.

In Fig. 1, Plate LIV, four additional men bring the figures up to 3 601.7 lb. total and 100.0 lb. per sq. ft.

In Fig. 2, Plate LIV, thirteen additional men, making thirty-seven in all, bring the results up to 5 552.5 lb. total and 154.2 lb. per sq. ft. The average weight of these thirty-seven men is seen to be 150.1 lb.

* *Engineering News*, Vol. LI, p. 360.

Mr. Johnson. In Fig. 2, Plate LIV, no attempt was made to reach a maximum, but only a full 150 lb. per sq. ft., and a number of the men testified that the congestion seemed to be materially less than that to which they are subjected upon the drawbridge above referred to. Obviously, there are several very short men in the picture who could be replaced by taller men occupying little or no more room, and it seems to be clear that 160 lb. per sq. ft. is quite within the possibilities.

Fig. 3, Plate LIV, is another view of a crowd under which the average floor load is 41.8 lb. per sq. ft. The five men shown are in an alcove, 4 ft. square, and their combined weight is 669 lb.

In the light of these experiments, the writer is convinced that 80 and 100 lb. per sq. ft. are of common occurrence throughout whole aisles and passageways, and even 125 lb. cannot be infrequent. The writer knows of grand stands where 3.3 sq. ft. is the allowance per person seated. This, assuming 150 lb. as the average weight per person, would make 45 lb. per sq. ft., with no allowance for the weight of the seats themselves.

It is freely admitted that the writer's results give figures greatly in excess or those given by the accepted authorities (outside of some municipal building laws), both in the United States and in Europe, but the experiment is one very easily tried by anyone who may feel unconvinced.

Doubtless, mixed crowds of men and women, such as football spectators, may weigh less per square foot, with an equal degree of personal discomfort, than the body of students in the writer's experiments.

It should be remembered that a closely packed crowd is not likely to be in a mood to take calmly any undue deflection or appearance of weakness in the floor, and the result of such seeming insecurity is not pleasant to contemplate. In the writer's opinion, such floors as those of passageways, corridors, standing-room in theaters, assembly rooms without fixed seats, ballrooms, etc., should be calculated for a weight closely approaching 150 lb. per sq. ft., or, in some cases, even more, without exceeding the unit stresses of Mr. Schneider's Paragraph 17. Possibly, a large standing assemblage, such as is common at political meetings, likely to applaud by stamping; or, a throng of dancers; or a body of drilling solders, might call for an additional impact provision. Moreover, it should not be forgotton that in an assembly room "with fixed seats" those seats are sometimes removed in order to accommodate as many as can be packed into it standing.

To summarize briefly, the writer begs to maintain:

I.—That the extreme value of the statical load from a crowd of men is a very few pounds, if any, below 160 lb. per sq. ft.;

II.—That there are many structures which contain considerable areas where a load as great as 150 lb. per sq. ft. is to be expected occasionally and fully provided for;

Fig. 1.—Crowd Weighing 3 601.7 Lb., Total; or 100.0 Lb. per Sq. Ft.

Fig. 2.—Crowd Weighing 5 552.5 Lb., Total; or 154.2 Lb. per Sq. Ft.

Fig. 3.—41.5 Lb. per Sq. Ft. in Alcove.

Mr. Johnson.

III.—That these facts should be clearly stated, and that the maximum loads should not be left to be taken care of by a concentrated-load specification which might or might not provide for them according to the closeness of the beam spacing;

IV.—That the distributed-load values given in Table 2 ought, accordingly, to be increased materially, at least for ground floors of office buildings, assembly rooms, and staircases leading thereto, and in many cases for sidewalks.

The writer begs to record his indebtedness to N. E. Olds, one of his students, who took the photographs accompanying this discussion.

Mr. Macdonald.

H. P. Macdonald, Jun. Am. Soc. C. E.—The speaker wishes to take exception to Mr. Llewellyn's remarks about the inspection of castings. The obstacles to the inspection of castings when they first leave the sand are more those of the inspector's than the foundryman's making. The average inspector does not care to get around [illegible] or 5 o'clock in the morning when the pieces cast the day before are shaken out, and, as the foundryman has to use his flasks, he cannot wait until the inspector eats a late breakfast.

The speaker does not think that any one of the big foundries around New York City would raise the least objection to an intelligent inspector watching the manufacture of its product from the time the metal is charged in the cupola until it leaves the machine shop, but would rather welcome his advice and suggestions. An inspector who can be deceived, by the use of "Smooth On" or such compounds, into passing a defective casting, does not know his business, or is careless in his work.

A case of very excessive floor loading in an office building has recently been brought to the speaker's attention. In a room, 13 by 16 ft., were stored 1 500 000 pamphlets, weighing 25 lb. per thousand, besides a 1 600-lb. safe and sundry articles, which brought the total load to more than 40 000 lb., giving at least 200 lb. per sq. ft. of floor area.

Mr Goodrich.

E. P. Goodrich, Jun. Am. Soc. C. E. (by letter).—Mr. Schneider's restriction of his subject to buildings is important in connection with the formation of a specification for the design of pile foundations, as well as for that of other parts. It is inferred, as a further restriction, that his intention is to treat only structures in which a steel frame is to be found, so that the building may be expected to be a rather large one and the foundation loads of some size. Where this is the case the foundations are massive, and the effect of vibration may be considered *nil*, except perhaps in some machine shops and buildings of similar nature.

Mr. Goodrich. Even with pile foundations, unless the piles are driven to rock or other firm bearing stratum, it is important to proportion the areas of the foundations or the number of piles used in each, so that exactly equal settlement will be secured at all points. In order to do this, a proper relation must be established between the live and dead loads, proper values must be assumed for the bearing powers of various soils, and a correct pile spacing and supporting power must be assumed for the kind of earth and for the other factors entering the problem. Of course, none of these items can be determined beforehand with any great degree of accuracy or even that which approaches the degree of certainty now attained in steel design, and every means which is likely to diminish the possible or probable error should therefore be used.

The writer considers the method given by Mr. Schneider for the determination of a unit strain on foundations as perhaps the best that can be devised, except that it will be better to take, not a column with the greatest relative live load, but one with an average live load. In this way the percentage of possible variation would be cut in two. In the case of power plants, etc., it will often be better to build so broadly that practically no settlement will occur in any case, and to provide entirely separate foundations for all parts likely to be subjected to excessive loading. In warehouses, little or no reduction of load can be permitted.

The values assigned by Mr. Schneider for the permissible pressures on various soils are perhaps as close as may safely be determined empirically, but the values given seem to the writer needlessly conservative. Recourse should always be had to direct experiment wherever possible, and it is well to note that some settlement must always be allowed for; and that it is infinitely better to test a relatively large tract (from 4 to 6 sq. ft.) with the actual load expected, than to overload greatly a tract of only 1 or 2 sq. ft. Furthermore, this test should be made as near the actual base of the building foundations as possible, not at the surface of the ground, and the test should be made with filling packed around the bearing mass and not in an open hole. These methods may seem to be unnecessarily exact, but their value has been proven in the writer's experience.

The writer thinks Mr. Schneider's paragraph on "The Bearing Power of Piles" can be somewhat improved. It is understood that Mr. Schneider intends to omit from his specification all that part relating solely to construction work, except in so far as the word "workmanship" covers it. With this point in mind, the first objection is, that the design of the foundations and their construction have been confused. A designer must assume a certain unit bearing power, and dimension his foundation accordingly. The superintendent of construction has to see that the unit stresses assumed by the designer are actually to be provided for, whether they be for concrete, earth or piles. Therefore, it would seem that the including of the pile formula in a specification for unit stresses was putting it in the wrong place. The first two sentences of the paragraph in question are permissible, except that it would be better to specify that piles are to be computed as round-end columns whenever they are to be driven to rock or an equivalent bearing stratum, and as columns fixed at one end and of a proper length when driven under other circumstances. In sand, when the pile is entirely buried, 10 ft. is ample for such "proper" ength, and even in the mud on the bed of a river piles have been known to fail by breaking off at the mud surface when driven only 15 or 20 ft. into it. It is usually unnecessary to state that the bearing power of the pile itself is not to be exceeded, as column formulas always include that factor; and the usual stipulation as to a maximum allowable load per pile always precludes the possibility of its being even approached. The allowable end bearing assigned by Mr. Schneider for yellow pine is 1 500 lb. per sq. in. His maximum allowable load on a pile is 40 000 lb. A short yellow pine cylinder less than 6 in. in diameter would support this load, and 6 in. is the usual minimum size for the points of wooden piles. Mr. Goodrich.

The writer believes that the best practice is to assume a given load per pile, design all footing accordingly, and make the superintendent of construction provide and drive piles which will sustain this assumed load. In that case the designer's care will be to provide just the proper number under each footing and to space them so that each will develop its full proportion of the given load. To this end, groups should be made as nearly circular as possible, especially when they consist of any considerable number of piles. The corner piles of square groups of sixteen piles might just about as well be omitted. It is of the utmost importance not to space piles too closely together, or, if close spacing is necessary, to drive all to such depth that the bearing power of the earth at that depth is sufficient to provide the necessary supporting power. All the piles under a building should be driven to the same depth, if possible, and the areas of groups should be carefully proportioned to the loads carried, unless the spacing is great enough for each pile to develop its full supporting power independently. Tests* made by the Department of Docks and Ferries, of New York City, prove conclusively that piles driven in the North River mud, even to considerable depths, influence each other to some extent when 6 ft. apart, and are practically a unit in their action when only 3 ft. apart. A group of two piles thus spaced had a supporting power of only about one and two-thirds times what a single pile developed when properly spaced.

*See "Wharves and Piers," by John A. Bensel, M. Am. Soc. C. E., Papers.—International Engineering Congress, 1904.

Mr. Goodrich. Under the circumstances, it seems better to devise some rule for the spacing of piles under foundations, make each pile carry an equal load, and drive all to the same depth. If the earth is uniform in character, this depth seldom need be very great, but, if it is not uniform, the piles should be driven through the variable stratum if possible. Under such circumstances, no pile formula is needed, because it is only necessary to drive until a specified penetration is attained in each case under a standard height of fall of hammer. From the writer's experiments* he believes that a penetration of 1 in., when produced by a 2 000-lb. hammer falling 15 ft., as freely as possible with rope attached, will give most satisfactory results. With hammers of different weights the same drop should be maintained and a corresponding penetration depended upon. Higher falls are much less effective. In some cases penetrations amounting to as much as 30 in. have proven satisfactory under conditions which will be discussed later. The experiments, and actual tests made, tend to show a supporting power of 100 000 lb. for such a pile when acting alone and free to develop the full effect of the blow producing a 1-in. penetration. Theoretically, a pile would have to be driven about 50 ft. into earth having a high ratio of lateral to vertical pressure (such as silt), in order to attain a frictional resistance of this amount, and to a depth of about 40 ft., to reach a stratum at which bearing and frictional resistances combined would develop this amount. With dryer soils and those containing much coarse and fine sand mixed, the depth, theoretically, would not be as great, it being slightly more than 30 ft. These depths are given on the assumption that the materials through which the piles are driven are practically homogeneous, and ignoring the considerable increase in supporting power which always occurs as soon as the piles are left to stand without interference even for a few hours. In the case of moist sand, this increased power often amounts to nearly double the original value, and, under certain circumstances, in the case of mud, to as high as ten times the original amount. This increase in supporting power is due to the settling back against the side of the pile of the earth disturbed in driving. This settling action is often very rapid, as evidenced by the difference in the effect of steam and gravity hammers. In fact, the writer believes the rapid blows of the quick-acting steam hammer are more than twice as efficient as the more measured ones (of equal theoretical effect) of gravity hammers. For these reasons, it is evident that a pile formula cannot be used indiscriminately, and that the actual supporting power of the soil must be taken into account in most cases.

With these ideas in mind, the maximum of 40 000 lb. proposed by

* "The Supporting Power of Piles," by E. P. Goodrich, *Transactions*, Am. Soc. C. E., Vol. XLVIII, p. 180.

Mr. Goodrich.

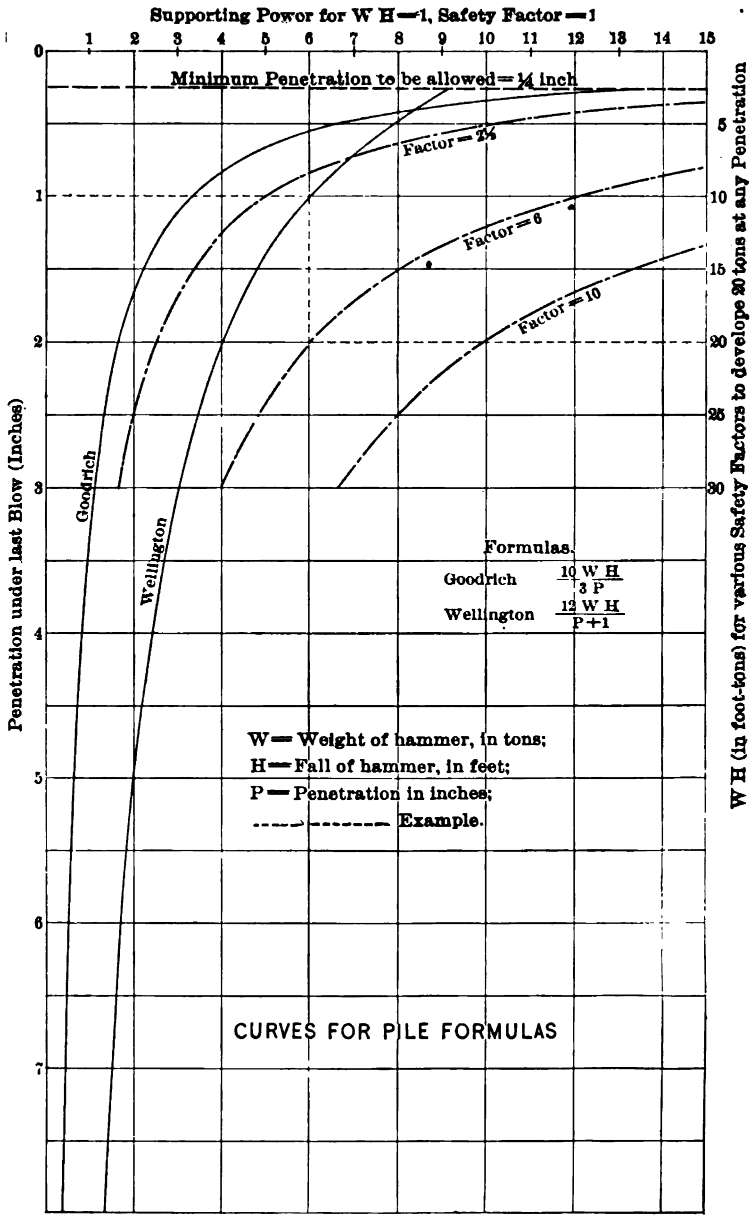

Fig. 5.

Mr. Goodrich. Mr. Schneider seems very conservative, giving a factor of safety of 2½ with the specification proposed by the writer and according to his formula, and a factor of 4½ if tested by the *Engineering News* Formula, without the factor of safety introduced. Wellington proposed the almost universal introduction of a factor of safety of 6, and it is introduced in the formula given by Mr. Schneider. It must be remembered that all formulas properly apply to piles only at the time of driving, with the probability of the above factors of safety being doubled with lapse of time with most soils, and, in many cases, with the possibility of increasing them four-fold.

Under any circumstances, it is not necessary to drive piles harder than just enough to develop a supporting power of two and one-half times 40 000 lb., if that be specified as the proper factor of safety and maximum allowable load per pile.

Fig. 5 shows the curves of maximum supporting power of the *Engineering News* (Wellington) Formula and of that of the writer, for different penetrations, on the assumption that $WH = 1$; it also gives a ready means of finding the value of WH, the factor of safety, or the penetration, when the other two terms are assumed, and a supporting power of 20 tons is desired.

A value of WH of 15 ft.-tons is about as small as it is economical to use in work of any magnitude, and a value of 30 ft.-tons is far above good practice. It is thus seen that penetrations of less than ¼ in. are little used, and, in any case, the writer would absolutely exclude those less than ½ in., and prefers to use only those amounting to 1 in. or more. This restriction, together with that as to a maximum load, reduces practically all pile formulas, for their curves, between the limits given, to almost parallel ones, so that results differ only in the factor of safety which the author purposely includes or unwittingly introduces in the making up of his formula. On this point there is almost as much difference of opinion as there is difference in formulas.

Thus, it seems altogether best to exclude all formulas from incorporation in a specification of "unit stresses" and, under "workmanship," simply to state the conditions required to afford the assumed stresses, basing the conditions on the best experimental evidence.

If it be assumed that the angle of internal friction of earth has a tangent of 0.4, the allowable spacing from center to center which will develop a bearing power of 40 000 lb. per pile with a factor of safety of 2½, at the depths to which the pile is driven, is given by the following table:

Depth to which driven..........	10	15	20	30	40	50	ft.
Minimum spacing necessary to develop load...............	4.0	3.3	2.7	2.2	2.0	1.8	ft.

It is to be remembered, however, that it is not possible to drive piles too closely into earth, because the latter has only a limited compressibility. With the spacings given above, the theoretical actual increase in density of the earth, if 12-in. cylindrical piles be driven, is as follows: Mr. Goodrich.

Spacing................	4	3.3	2.7	2.5	2.2	2.0	1.8 ft.	
Percentage of increase in density of earth after driving.............	5	10	12	15	21	25	33	per cent.

Earth with 35% of voids, if compressed so that all voids are filled, will increase in density only 54 per cent. From quite a number of tests of the compressibility of soils, made by the writer, it is evident that a tremendous amount of energy is wasted in pile driving if the piles are spaced so closely that any great compressing of the soil must be done. This wasted energy is not disclosed in any pile formula, and serves to give exaggerated values when such formulas are applied. Considerable practical experience also confirms this and all the other theoretical results given above. Thus, it is evident that, even with piles spaced 2½ ft. apart, the amount of compression suffered by the earth is more than one-quarter of the maximum possible amount in many cases, and that considerable energy must be wasted in driving so closely. A spacing of 3 ft. is much to be preferred, especially when it is seen that the theoretical depths to which it is necessary to drive the piles, in order to develop a safe bearing power of 40 000 lb., are 16 ft. for the 3-ft. spacing and 26 ft. for the 2½-ft. spacing. The writer thinks that a minimum spacing of not less than 2.7 ft. should ever be allowed and that 3 ft. should be used wherever possible.

The writer therefore proposes the two following paragraphs on "Piles," in place of the one given by Mr. Schneider.

Unit Stresses.—All pile foundations are to be designed so as to bring, as nearly as possible, a load of 40 000 lb. on each pile. Piles are to be spaced not closer than 2.7 ft. from center to center, and all groups are to be made as nearly circular in general outline as possible. Piles are to have such diameters as will afford ample stiffness and give sufficient area to act as columns, considered as pin-connected when driven to rock or equivalent bearing at depths less than 20 ft., and considered as fixed at the lower end and of 20 ft. length under all other circumstances.

Workmanship.—All piles are to be of lengths and diameters not less than those specified, are to be spaced accurately as shown upon the drawings, and driven as nearly as possible to a uniform depth, provided uniform penetration is developed under equal blows of the hammer. Such penetration is to be 1 in., as nearly as possible, under a 15-ft. blow from a 2 000-lb. gravity hammer, or a proportional penetration under any other weight of hammer with the same fall. Only

Mr. Goodrich. piles of length necessary and just sufficient to develop this penetration are to be driven. Should a steam hammer be used, equivalent values of the hammer weight and the height of fall of a gravity hammer are to be used, and a penetration of 2 ins., as nearly as possible, is to be secured.

Mr. Ketchum. M. S. KETCHUM, ASSOC. M. AM. SOC. C. E. (by letter).—This is a valuable and timely paper, and it is to be hoped that, together with the discussion, it will lead to more rational methods for the structural design of buildings. The author has covered the entire field very thoroughly, and, for the most part, the specifications meet with the writer's approval.

Without attempting to discuss the specifications as a whole, the writer would call attention to the following paragraphs:

Paragraph 7.—The weights of roofs and roof coverings vary so much that it would appear to be more logical to calculate the weight of trusses, purlins, sheathing and roof covering in each case. For calculating the weight of roof trusses for mill buildings, train-sheds, etc., the following formula has been proposed by the writer:*

$$W = \frac{P}{45} A L \left(1 + \frac{L}{5\sqrt{A}}\right)$$

in which W = weight of roof truss, in pounds;

P = capacity of the truss, in pounds per square foot of horizontal projection of roof;

A = distance from center to center of trusses, in feet; and

L = span of truss, in feet.

Paragraph 8.—In localities subject to snowfall, it would seem desirable to consider a minimum ice or sleet load, of say 10 lb. per sq. ft., which would be on the roof at the time of maximum wind. The author appears to have had the sleet load in mind in specifying a minimum snow load of 10 lb. per sq. ft.

Paragraph 17.—The writer would think it more rational to specify the allowable shear on the net section of webs of plate girders.

Paragraph 22.—Cross-bending and direct stress should be combined by the application of a rational formula which takes account of the fact that transverse loads produce larger stresses in compression members than in tension members. The writer has used Johnson's formula,†

$$f = f_1 + f_2 = \frac{M_1 Y_1}{I \pm \frac{P l^2}{C E}} + \frac{P}{A},$$

for combined stresses, and believes that it is the most satisfactory formula yet proposed. The proposed reduction of the transverse

* "The Design of Mill Buildings and the Calculation of Stresses in Framed Structures," Engineering News Publishing Co., New York.

† "Modern Framed Structures."

bending moment does not appear to be proper, except in the cases of wind moment, moment due to weight, and moment due to eccentric loading. The writer believes that the usual method, of increasing the allowable stresses by 25% when wind is considered, and by 10% when weight and eccentric loading are considered, is to be preferred. Mr. Ketchum.

Paragraph 37.—The writer has noted with pleasure that the author has adopted the stresses and the straight-line formula for the design of steel struts and columns proposed by the American Railway Engineering and Maintenance-of-Way Association. Under the circumstances, however, it does not appear to be consistent to use a straight-line formula for designing steel members and a curve formula for designing timber struts and columns.

The straight-line formula adopted by the Cities of Philadelphia, Buffalo and Minneapolis, and used by the writer in his specifications for "Mill Buildings,"*

$$p = C - \frac{C}{100}\frac{l}{d},$$

in which p, C, l and d are the same as used by the author, would appear to be more in keeping with the spirit of the specifications.

Paragraph 62.—This clause is a decided improvement on the usual clause specifying 1¼ in. for all sizes of rivets.

Paragraph 66.—The writer believes that one-sixtieth of the distance between rivet centers will give batten plates which are too thin, and that the usual specification of one-fortieth should be substituted.

Part II, Paragraphs 2 and 3.—The use of Bessemer steel is allowed in Paragraph 2, but it is virtually cut out by the requirements in Paragraph 3. The writer favors specifying that steel shall be made by the open-hearth process except for temporary or unimportant structures.

George H. Blakeley, M. Am. Soc. C. E.—The high authority of the author will undoubtedly commend the proposed specification to those seeking guidance in the structural design of buildings, and, also, undoubtedly, will influence greatly the design of such work. The requirement of the consideration of concentrated loads in the designing of floors is a commendable provision which deserves the attention of those who have not already given the matter the attention that the importance of the subject warrants. Mr. Blakeley.

The loadings proposed by the author, however, should be considered carefully before general adoption, as it is a serious question whether they do not produce an asymmetrical design, making the floor joists heavier than necessary and the girders lighter than desirable, within the proper margin of safety.

* "Steel Mill Buildings," Engineering News Publishing Co., New York.

Mr. Blakeley. If it is proper to provide for supporting a 5 000-lb. safe, then, only in exceptional cases could the entire weight of the safe be carried on a single joist. Such a safe would occupy an area of about 3 by 5 ft., and it is proper to consider such a distribution of the load in designing. With joists of 15 ft. span and spaced 5 ft. apart, it is impossible to place such a safe in any position where it would produce a loading of a single joist in excess of that caused by a center load of 3 500 lb. As the proposed specification does not purport to be a simplification of calculation, it would be proper to specify a definite area covered by the concentrated load, instead of considering it under the impossible condition of being concentrated at a mathematical point. Such a modification would produce, in general, a reduction of the sizes of floor joists, and without impairing the adequate carrying capacity.

On the other hand, it is questionable if the specification of a live load of 1 000 lb. per lin. ft. for girders is sufficient to cover the contingencies of loading that may occur in buildings. For example, a perfectly possible case is that of a wing of an office building with two rooms, each 16 ft. square and with the girder under the partition between the rooms. This girder supports a floor area of 256 sq. ft. It is possible that the occupant of each office may have a heavy safe, which he would not place against the door partition nor against either of the two outer walls of the offices, as it might interfere with the windows, but each would place his safe against the partition wall over the girder, in which case there would be a concentration of two safes on the girder. If these were 5 000-lb. safes, then the girder would be loaded by the safes alone equivalent to a uniform load of 1 250 lb. per lin. ft., or 25% in excess of the load for which the girder was designed, and without any further provision for carrying the 256 sq. ft. of floor area which must still be supported by the girder. Of course, with certain arrangements of the floor framing, with due regard to the area of floor space occupied by the safes, and with precise calculation, the effect of the loading produced by these safes would be very much reduced, and might be even as low as an equivalent of 750 lb. per lin. ft. of girder. But, under a possible arrangement of floor framing, these safes, with due regard to their area of floor space and with precise calculation, would produce an equivalent loading of 1 000 lb. per lin. ft., thus consuming the entire carrying capacity for which the girder was designed and without leaving any remaining provision for carrying the floor space which still must be supported by it. It is quite certain that the authorities in charge of the building would direct the safes to be placed where they would be supported by the girder, though they might direct that they be placed at the wall end or at the column end of the girder, which would materially lessen the effect of the loading. In many buildings, however, there is no intelligent supervision of these matters, and the disposition of safes is left largely to the convenience of the tenants. Mr Blakeley.

In the case of office buildings occupied by lawyers, it is possible to have bookcases filled solid with books from floor to ceiling, and on each side of the partition over a girder, producing a load of from 400 to 450 lb. per lin. ft. of girder. Such offices are usually of fair size, and, after deducting the effect of the bookcases, there may be left in the girder a carrying capacity of less than 30 lb. per sq. ft. of the floor of the offices. Such offices at times may be fairly crowded with people, as in the case of an important hearing before a referee, and may have a floor load of at least 50 lb. per sq. ft. caused by a crowd of people at such a time.

In the case of store buildings, a live load of 1 000 lb. per lin. ft. for girders is insufficient to provide for the conditions of loading that will prevail in such buildings. In the construction of store buildings, the tendency is to space the columns far apart, and 20 ft. from center to center is not unusual. For such a case, with joists spaced at 5 ft. centers, according to the proposed specification, the joists would be designed for a live load equivalent to 160 lb. per sq. ft., while the girders would be designed for a load of only 50 lb. per sq. ft. On a limited area in such buildings it is not unusual to have a crowd of people equivalent to at least 80 lb. per sq. ft., especially on bargain days and during the holiday season. It is perfectly possible that on such occasions areas affecting a girder in such a building will be loaded considerably in excess of 80 lb. per sq. ft., as against the 50 lb. per sq. ft. provided for by the proposed specifications. Moreover, portions of store buildings at times partake of the character of light storage buildings, in the receiving and shipping of goods. Crockery and glassware in crates, set side by side and not piled, will produce a load of 120 lb. per sq. ft. Flannels in cases, piled 4 ft. high, produce a floor loading of 100 lb. per sq. ft. Cotton prints in cases, set side by side and not piled, produce a floor load of 93 lb. per sq. ft. Woolen dress goods, in cases set side by side and not piled, produce a floor load of 84 lb. per sq. ft. Brown sugar in barrels, set side by side, produces a floor load of 113 lb. per sq. ft. These and other articles handled in store buildings will at times accumulate over certain areas and fully load the girders; therefore, in the design for such buildings, the possibilities of such loadings should be considered and the girders be designed accordingly.

It is reasonable that the live-load carrying capacity of girders should have some relation to the floor area which they are to support, but, according to the proposed specification, girders spaced at 25-ft. centers would have no more live-load carrying capacity than girders of the same span spaced but half the distance apart, or at 12½ ft. centers. According to the proposed specification, each girder would be designed

Mr. Blakeley. for a live load of 1 000 lb. per lin. ft., notwithstanding the fact that one of the girders would be called upon to support a floor area twice as great as the other. It is to be questioned if such a design will provide for the possibilities of loading which may occur.

The concentrated-load method of designing floor framing is commendable, but the concentrated load should have a specified area over which it is distributed, and such distribution should be considered in the design of the joists and the girders. It is probable that no girder should be designed for a load of less than 1 000 lb. per lin. ft., but, on the other hand, it does not seem advisable to design any joist or any girder for an office building, or for a store building, for a load less than 80 lb. per sq. ft.

The reduction of live loads on columns is in more or less general use, but it is questionable if it is proper to consider any reduction of column loading in warehouse buildings. It is not quite clear that the proposed specification sanctions such a reduction of column loads for buildings of this type,

In selecting the proper live loading in the design of buildings, too much consideration should not be given to the question of probability, but due and proper regard should be given to reasonable possibilities of loading which may occur. A specification for general use should be very carefully framed in this respect, and, in the design of a building, should not leave an opening for work which might prove to be inadequate for reasonable possibilities of loading.

Mr. Clermont. JOHN B. CLERMONT, ASSOC. M. AM. SOC. C. E.—It is wise, on the part of the designing engineer, in proportioning a structure, to consider that there is something more than low theoretical live loads, in designing office buildings, churches, theaters, halls and other public buildings, especially in cases where alterations may be considered as probable.

In a certain case, alterations in an office building involved the moving of a large steel vault, which had been erected on the second story and supported on brick foundation walls to bed-rock. This vault had to be transported over the floors of a portion of the old building and a portion of a new building adjoining. Its weight was about 12 tons, and the floors of both buildings were designed for a live load of 150 lb. per sq. ft.

In another part of this building, also on the second story, some changes were made in the steel construction, increasing the sizes of beams and girders and strengthening the supporting columns, in order to support another vault, weighing about 246 tons when complete. All parts of this vault were in sections, excepting the vestibule and door, and these weighed 17 and 10 tons, respectively. These pieces had to be transported over a section of the regular framing for a distance of about 40 ft. It was again found very advantageous to have a floor construction designed to carry a live load of 150 lb. per sq. ft. Mr. Clermont.

While the two foregoing cases may be considered in a measure as extremes, specific cases of overloading in public buildings are of frequent occurrence. These may be caused in various ways, such as overcrowding of persons in small spaces during fire or panic; tenants in office buildings securing storage space and loading the floors with paper, bulky sample goods, records, etc., as high as the ceilings will permit; in offices which are used for show rooms where heavy case goods are constantly handled and often stacked high for lack of space; in fact, it seems almost impossible to foresee the numerous variations in live loads possible in all kinds of public buildings.

The variations in live loads, amounting to as much as 100% in the building laws of different centers of population, mentioned in Mr. Schneider's paper, are easily accounted for on the ground of their vastly different requirements, but as to the conditions of their commerce and population and because each is based upon local experience. In consideration of this, the establishment of a uniform standard for live loads, in order to harmonize their constructions, would mean to do no less than harmonize their conditions, and would not be conducive to the best results. By this it is not meant that the disparities between their different requirements by law are not subject to a general system of revision for similar requirements, but that the difference between the accepted good practice of these several places and the acceptance of a uniform maximum standard live load of 40 lb. per sq. ft., would be a step in the wrong direction, and, with few exceptions, for all public buildings, a live load of less than 100 lb. per sq. ft. ought not to be considered.

Attention has been called to the shifting of cores in round cast-iron columns. In an office building erected in New York City about twelve years ago, and well inspected during erection, it was found before its completion that one of the interior columns on the first story was cracked for a distance of about 5 ft. below the shelf brackets at its top. This was a case of core shifting directly in the line of the joints of the moulds. The crack was straight down along the joint mark, and, while the column was of ample section to support the load, it was necessary to place stout wrought-iron straps around it. Recently, these straps had to be removed in order to make room for a marble covering, and, at the speaker's suggestion, the column was wound with light, flexible, seven-strand cable. This is an example of what may happen if the inspection of cast iron is not thorough.

Oscar Lowinson, Assoc. M. Am. Soc. C. E. (by letter).—The author's effort to establish a standard specification for structural work is deserving of the highest commendation, and its success will be demonstrated by its adoption, with such modifications as more detailed experience than the author possesses in parts of the field which he Mr. Lowinson.

Mr. Lowinson. has covered, will be a tribute to him for having brought it forward as a standard of reference. The following comments and suggestions are made in reference to some of the matters specified, and the hope is expressed that the compiled results of the discussion will be used as a standard to be changed only by reason of changing conditions or increased knowledge.

In the first place, the author is warned that his live loads for dwellings, hotels and apartment-houses are too small. Take a dwelling, for instance. In the life of every family there occur periods during which the apartments are crowded. Engineers are compelled to design buildings to meet the most unfavorable conditions of loading, and must be prepared for, not only a crowd, but a crowd stamping at the same time, which causes vibration in the building and must be provided for. The writer suggests for dwellings a minimum live load of 65 lb. per sq. ft. In country residence work, it is the writer's practice to design the first floor for a live load of not less than 80 lb. per sq. ft., and in city houses the sizes of the beams and girders are usually determined by their deflection, which should not exceed $\frac{1}{30}$ in. per ft. because of the danger of cracking plastered ceilings.

In the case of hotels, the lobby and such rooms as may be occupied for public purposes should be placed in the same class as assembly rooms.

The author separates assembly rooms into two parts, those with fixed, and those with movable, seats. This is questionable practice, for it is frequently the custom, in New York City theaters and assembly rooms, to lay a secondary floor over the seats. The writer recommends for such buildings a loading of 125 lb. per sq. ft., his reason being that, under crowded conditions, such as during political meetings, the live load frequently amounts to 100 lb. per sq. ft., and the vibration caused by stamping may easily increase this to the equivalent of 125 lb. Further, in view of Professor Johnson's recent experiments, wherein he obtained even greater loading in crowds, the writer believes the loading adopted by the author to be too light.

Stables and Carriage-Houses.—Stables and carriage-houses should be designed for automobile loads. The writer weighed some automobiles a short time ago, and found that a carriage automobile weighed 4 000 lb., with a concentrated loading of 1 500 lb. on a wheel. The writer would design a private stable in accordance with the loading given by the author, but for the carriage-house of a stable where trucks might be stored he would assume a greater load, his New York City practice being to design such a floor for a live load of 250 lb. per sq. ft. The stalls he would design in accordance with the author's figures.

Sidewalks.—It has been the writer's practice to design sidewalks for a live load of 350 lb. per sq. ft., and he would suggest that the distributing load be made equal to that figure.

Warehouses and Factories.—The writer has frequently been called upon to determine the weights on warehouse floors, and has found loads of 350 lb. per sq. ft. and greater. He would not recommend less than 250 lb. where either paper or iron is to be stored. In fact, a storage building will frequently suffer because of this. Mr. Lowinson.

Though hardly pertinent to this discussion, an instance may be cited where a collapse occurred in a warehouse used to store barrels. Owing to vibration in the building, caused by passing trucks, the barrels became wedged, and threw a bearing wall out into the street. Wedging of this kind will concentrate at times an enormous load on a single section of floor.

Office Buildings.—This is an age of great and quick changes, and, already, some of the older high office buildings are being converted into storage buildings, with loads far in excess of those for which they were designed; and, although the writer would be satisfied with a distributed loading of 65 lb. per sq. ft. if he were sure the building would never be used for any other purpose, he thinks a provision of 150 lb. per sq. ft. not at all too large.

Wind Pressure.—The writer believes that a clause should be inserted providing that, where the walls are other than curtain walls, and the skeleton does not proceed more than three stories in advance of the walls, temporary wind bracing (wire cables, etc.) should serve. In all possible locations, stiff knee-braces should be insisted upon at all column connections.

Foundations.—The writer would separate wet sand from soft clay. Quicksand, for instance, should never be used on which to found. Wet sand will frequently bear from 4 to 6 tons, as long as it is confined, and a restriction to 1 ton should not be made absolute. On the other hand, the writer would hesitate long before permitting a load of 6 tons on any but the hardest kind of gravel, and then only when it overlies rock. It is his practice to permit a load of 15 tons per sq. ft. on Portland cement mortar, and he allows only 10 tons per sq. ft. over Portland cement concrete.

Pressure on Wall-Plates.—For the pressure on wall-plates, the writer would use 200 lb. instead of 250 lb.

Shrinkage and Masonry.—Great trouble has been experienced with stone facings on brick walls because of the unequal shrinkage of the brickwork and the stonework; for this reason the writer suggests a class wherein stonework must be considered either as non-bearing, or it should pass entirely through the wall at certain distances in its height so as to get a proper bond. In case of shrinkage, in the former instance, the facing is held by galvanized-iron anchors.

Paragraph 37.—Timber Columns.—It is suggested that a more modern formula than the Gordon formula be used for timber columns.

Mr. Lowinson. *Details of Floor Beams.*—The writer would add that where a floor beam transmits a heavy load it should rest upon a girder if possible, instead of framing it on the web. Where the girder is composed of two or more rolled sections, and unless definite means are taken to transfer the load from one to another, the load should be applied on top if possible.

These specifications will serve very well as a standard for structural engineers. The writer would be pleased to see, included with such a set of standards, standards of practice in such matters as framing timbers, cutting stonework, differentiation of skeleton, cage and independent masonry wall construction, protection and preservation of materials of construction, other than those included by the author, details of construction on piling, and protection against discoloration, efflorescence, etc.